PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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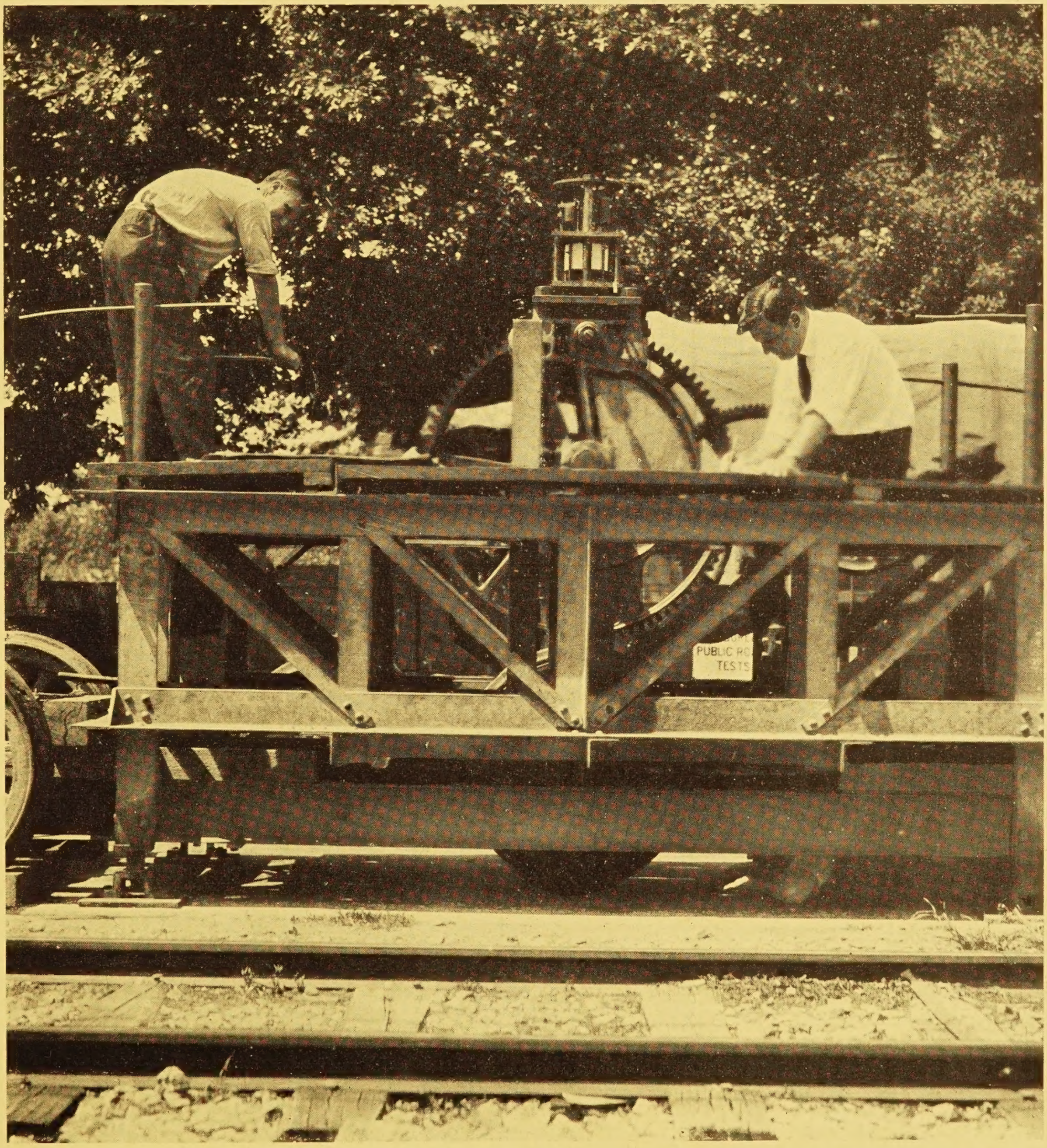

INVESTIGATING EFFECT OF PAVEMENT TYPE ON IMPACT REACTION

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VOL. 9, NO. 6 | AUGUST, 1928 | R. E. ROYALL, Editor

TABLE OF CONTENTS

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EFFECT OF PAVEMENT TYPE ON IMPACT REACTION

Reported by J. T. THOMPSON,[1] Highway Research Specialist, Division of Tests, United States Bureau of Public Roads

THE question has been repeatedly raised as to whether or not certain types of pavement or of pavement surfacing possess inherent cushioning or shock-absorbing properties, by virtue of which these types function to protect themselves or (when used as a surfacing) the bases on which they are employed.

In order to add to the rather meager data available on this subject, the Bureau of Public Roads carried out the investigation described in this report. Table 1 shows the character and location of the types tested.

TESTING APPARATUS DESCRIBED

The testing apparatus consisted of an impact machine [2] for producing the necessary impact forces and an accelerometer [3] for determining these forces. Both have been described in detail in previous articles, and only a very brief description of each will be given here.

The impact machine consists of a plunger the lower end of which is equipped with the rear wheel of a conventional motor truck, carrying, in this case, dual worn solid tires. The plunger can be raised against the action of a calibrated, conventional truck spring by cams actuated by a hand-wheel through a suitable gear train. When the plunger, with the truck wheel, has risen to the desired height of drop, it is released from the cam, the truck wheel being forced down upon the pavement under the combined influence of gravity and the truck spring. The height of drop of the plunger and the spring pressure can both be varied by means of suitable controls.

The accelerometer used is of the coil-spring type and consists of a weight mounted upon a vertical plunger, both of which are supported by a coil spring. When the mass to which the instrument is attached is decelerated, the weight and plunger of the accelerometer are forced downward, compressing the coil spring. The deflection of this spring is a measure of the deceleration. The coil-spring deflection is recorded upon silicated paper by a brass stylus mounted upon the accelerometer plunger.

In order to measure the permanent deformation produced in bituminous pavements by the impact machine, a crude but very satisfactory profilometer was devised. It consisted of a straight piece of wood about 2 feet long by 3 inches wide, upon which a piece of silicated paper was fastened with thumb tacks. Through the spot to be tested a line was drawn upon the pavement perpendicular to the plane of the wheel of the impact machine and broad-head nails were then driven into the pavement on this line and so spaced as to come under each end of the profile board, supporting it in a level position. The original profile was then recorded upon the silicated paper by means of a piece of stiff copper wire which was so bent that when it was held vertically with one end of the wire upon the line drawn upon the pavement the other end would make a mark upon the paper. The section was then tested, and after the testing machine had been moved away the deformed surface of the pavement was also recorded upon the profile board. The presence of both the original and final profiles upon the same piece of paper made it possible to determine how much and in what manner the pavement had deformed.

In addition to the regular impact-machine tests made with the apparatus described in the foregoing, some supplementary motor-truck impact tests were also conducted. The apparatus or equipment used in these tests, as well as the manner in which the tests were made and the data secured, are discussed in the latter part of this report.

IMPORTANT CORRECTION

By inadvertence a final paragraph was omitted from the article entitled "The effect of the length of the mixing period on the quality of the concrete mixed in standard pavers," published in the July number of Public Roads.

In the last paragraph of the article as published it was stated that—

"Summarizing the situation in the light of the data collected during this study, the evidence strongly indicates that where standard 21E and 27E pavers which are in good condition are used, neither strength nor uniformity of test results is improved by mixing the concrete over 45 seconds."

This statement properly summarizes the evidence obtained in the investigation upon which the article reported, but it might be inferred from it that the Bureau of Public Roads advocates reduction of mixing time to three-quarters of a minute. There was no such intention. While it is true that the tests quite generally disclosed no appreciable difference in strength between concrete mixed for three-quarters of a minute and longer periods up to three minutes, it is also true that half-minute mixing failed to produce satisfactory concrete. It is apparent, therefore, that reduction of the mixing time to three-quarters of a minute would leave a very narrow margin of safety to allow for the normal irregularities of practical operation.

It may also be questioned whether the pace set by so short a mixing time can be regularly and consistently maintained by the hauling equipment and other units of the construction plant. Weighing the results of the tests against these considerations it was the intention to suggest in a final paragraph the adoption of a mixing time of one minute as the minimum time desirable in practical operation.

The omitted paragraph is as follows:

As three-quarter minute mixing provides an insufficient factor of safety for practical operations, and as it is doubtful whether construction processes other than the mixing operation itself can be speeded up sufficiently to take advantage of the marginal time fraction, it is recommended that a mixing time of one minute be adopted as the minimum consistent with assurance of reasonable uniformity and adequate strength of the concrete. The evidence is strong that thoroughly satisfactory concrete can be produced by (21E and 27E) pavers in good condition with a one-minute mixing period.

The editor regrets the omission.

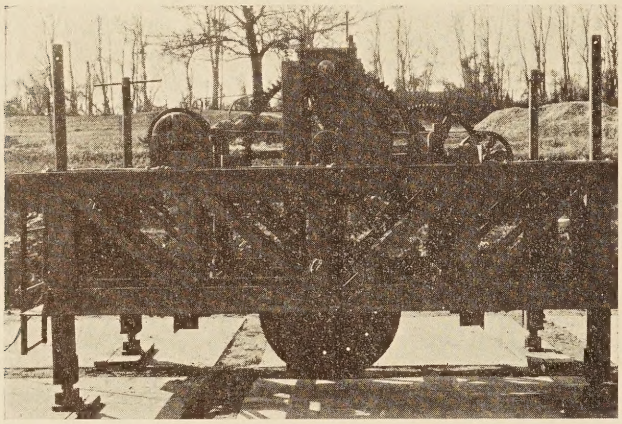
UNIFORM PROCEDURE ADOPTED FOR ALL TESTS

It was desired to develop for each type of pavement tested a characteristic curve showing the relation between the height of the drop of the wheel and the

[1] Also associate professor of civil engineering, The Johns Hopkins University.
[2] Teller, L. W., Impact Tests on Concrete Pavement Slabs, Public Roads, vol. 5, No. 2, April, 1924.
[3] Teller, L. W., Accurate Accelerometers Developed by Bureau of Public Roads, Public Roads, vol. 5, No. 10, December, 1924.

Table 1.—*Type and location of pavements tested*

Type No.	Number of points tested	Description of type	Name and location
1	12	2-inch Topeka; 6-inch 1 : 3 : 7 concrete base	Connecticut Avenue, Chevy Chase, Md.
2	12	2-inch bituminous concrete (District of Columbia specification); 6-inch 1 : 3 : 7 concrete base	Do.
3	6	3-inch combined sheet asphalt and binder; 6-inch 1 : 3 : 7 concrete base	Connecticut Avenue, District of Columbia.
4	6	3-inch combined sheet asphalt and binder; 7-inch 1 : 3 : 6 concrete base	Do.
5	3	1-inch sheet asphalt; surface-treated waterbound macadam base	Chevy Chase Circle, District of Columbia.
6	3	Surface-treated waterbound macadam	Georgetown-Alexandria Road, Arlington County, Va.
	3	6-inch 1 : 2 : 4 concrete	Woodbine Street, Chevy Chase, Md.
	3	do	Williams Lane, Chevy Chase, Md.
	2	do	East Underwood Street, Chevy Chase, Md.
7	17 { 3	do	Thornapple Street, Chevy Chase, Md.
	2	do	Northampton Street, District of Columbia.
	2	do	Chevy Chase Parkway, District of Columbia.
	2	do	Nebraska Avenue, District of Columbia.
8	9 { 3	6-inch 1 : 2 : 3 concrete	Columbia Pike, Arlington County, Va.
	6	do	Test road, Arlington Experiment Station, Virginia.
9	2	7-inch 1 : 2 : 4 concrete	Connecticut Avenue, District of Columbia.
10	4	7-inch 1 : 2 : 3 concrete	Columbia Pike, Arlington County, Va.
11	10 { 4	8-inch 1 : 2 : 3 concrete	Do.
	6	do	Test road, Arlington Experiment Station, Virginia.
12	6	10-inch 1 : 2 : 3 concrete	Do.
13	4	2½-inch brick; ¾-inch sand cushion; 6-inch 1 : 1½ : 3 concrete base	Circular test track, Arlington Experiment Station, Virginia.
14	3	2½-inch brick; ¾-inch 1 : 4 sand-cement cushion; 6-inch 1 : 1½ : 3 concrete base	Do.
15	3	4-inch brick; ¾-inch sand cushion; 6-inch 1 : 1½ : 3 concrete base	Do.
16	3	4-inch brick; ¾-inch 1 : 4 sand-cement cushion; 6-inch 1 : 1½ : 3 concrete base	Do.



Showing Details of Impact Machine, But Without Special Equipment Used in These Tests

total impact reaction developed between the wheel and the pavement. It was decided to define "height of drop" as the distance through which the wheel dropped from the moment it was tripped off the cams until its rubber tire first came in contact with the pavement surface.

In accordance with this definition, the following technique was employed for testing bituminous-topped pavements. The impact machine was first leveled carefully so that the plunger would be truly vertical, and then the plunger was raised on the cams until it was just at the point of trip-off. A deflection of 1 inch was then produced in the truck spring of the impact machine, and this deflection exerted a pressure of 1,500 pounds upon the plunger. The entire frame of the impact machine, carrying with it the cams upon which the plunger still rested in trip-off position, was next lowered (by means of four screws at its corners) until the tires on the wheel just made contact with the pavement surface. This condition was called "contact." The relative position of the plunger and impact-machine frame was then determined by careful measurement.

All of these operations were carried out with the plunger supported upon the cams at trip-off position, the wheel not being permitted to rest upon the pavement until the first impact was delivered and then for only as brief periods as possible.

The impact machine and of course its cams, with the plunger still resting upon them in trip-off position, was next raised to the first height of drop—i. e., one-third inch. In order that the truck spring always be deflected 1 inch at the instant the wheel came in contact with the pavement, regardless of the height of drop, it was necessary that the ends of the spring and the surface of the pavement bear a fixed relation to each other. However, since the ends of the spring are connected to the frame of the impact machine, every time the frame was raised it was necessary to adjust downward the ends of the spring by an amount equal to the change in elevation of the frame, so as to maintain this fixed relation.

Having completed the setting of the machine for the one-third inch height of drop, the pavement was struck three blows from this height, an accelerometer record being obtained for each blow. The new relative position of the plunger and frame at contact was then determined. The difference between this position and that formerly recorded represented the depth of any permanent deformation which had occurred in the pavement surface.

In preparing for the second height of drop (two-thirds inch) the machine was raised an additional third of an inch by means of the corner screws and the spring adjustment compensated for the increased height of drop. Then the machine was lowered by means of the corner screws an amount exactly equal to the permanent pavement deformation. This compensation was usually small and seldom amounted to more than 0.04 inch for three blows, even at the maximum height of drop employed, 1⅔ inches. For subsequent heights of drop the procedure was exactly the same as that just described.

Upon rigid pavements without bituminous surfacing the procedure was similar to that just described, except that no compensation was made for permanent pavement deformation.

DATA ANALYZED TO GIVE RELATION BETWEEN HEIGHT OF DROP AND IMPACT REACTION

In preparing the following data the accelerometer records were first accurately measured. These readings were then multiplied by the calibration constant of the accelerometer, 2,514 feet per second^2 per inch of record, to obtain the deceleration which, when multiplied by the mass of the wheel-plunger combination, $\frac{1646}{32.2}$, gave the impact force due to the striking mass. To this was added the weight of the plunger, 1,646 pounds, and the residual spring pressure at the moment of maximum impact and resulting maximum tire deflection.

The foregoing operation may be expressed by the formula

$$F = M(a+g) + P$$

where

F = total impact force.
M = mass of the wheel-plunger combination.
a = deceleration recorded by the accelerometer.
g = acceleration due to gravity.
P = residual spring pressure.

In evaluating P for the different heights of drop a curve (fig. 1) was obtained experimentally between height of drop and tire deflection. The recorded tire deflections deducted from the 1-inch spring deflection at contact gave the residual spring deflection. The pressure corresponding to this residual spring deflection was secured from static load-deflection data for the same spring and plotted against height of drop. This

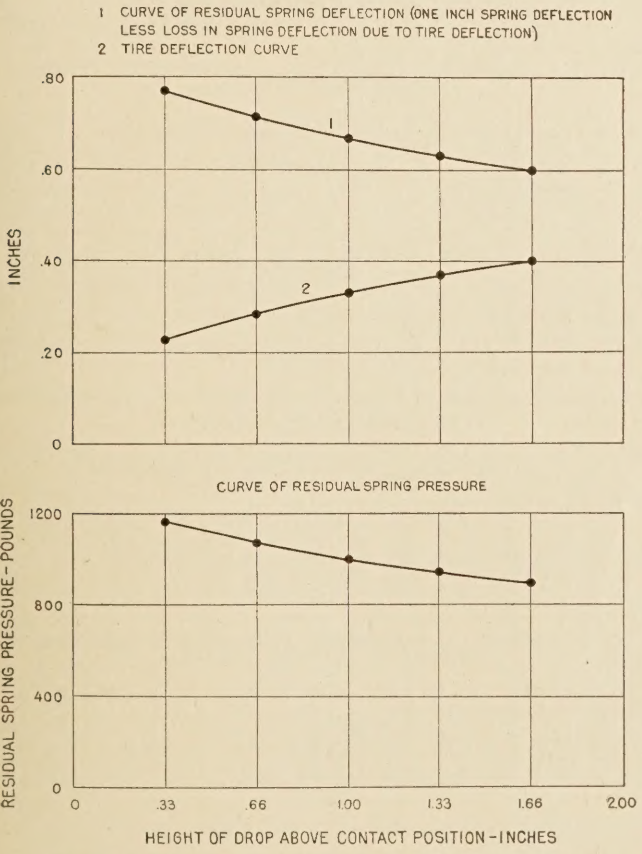
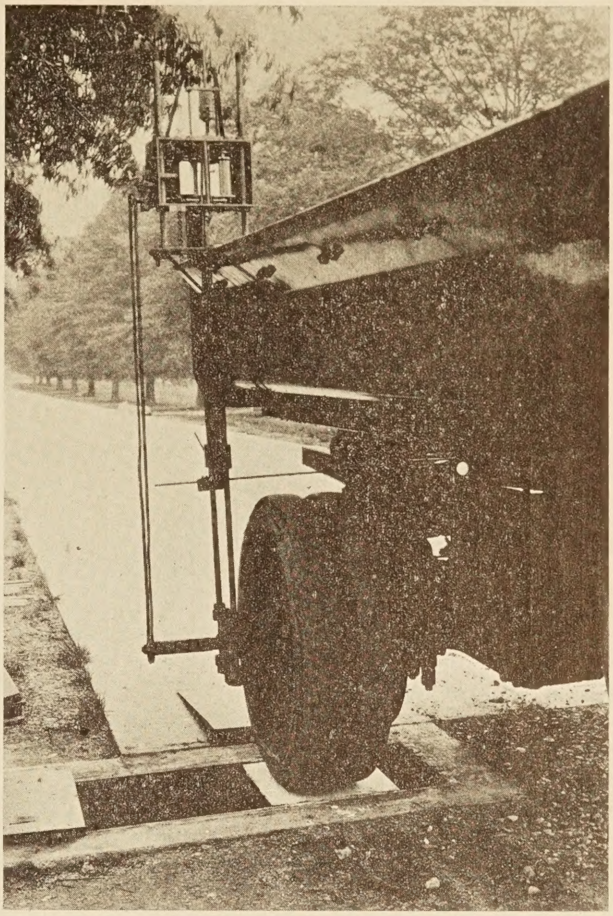


Fig. 1.—Curves Developed in Evaluating P



Showing Details of Accelerometer Mounted on Truck. This Picture was Taken in Connection with Another Investigation

curve was used in evaluating P. Table 2 shows a tabulation of all of the data collected.

CHECK TESTS INDICATE A SATISFACTORY DEGREE OF ACCURACY

At the beginning of the tests it was realized that the data would be worthless unless the settings of the machine could be so controlled as to be able to duplicate impact phenomena at a given test section for successive set ups. Accordingly, as a preliminary step, the machine was set up over a trial section of concrete at the Arlington Experiment Station and impact forces produced and measured for each height of drop. The machine was then replaced upon its chassis, towed away a few feet, and then moved back to the trial section and the test data redetermined. It was found for the same section that the impacts produced by the machine for identical settings could be duplicated and measured with a satisfactory degree of consistency.

Throughout the tests numerous check determinations were made, and near the end of the experiment the machine was brought back to the Arlington Experiment Station and a check test made upon the trial section referred to above. A close check was obtained, indicating that the performance of neither the machine nor the accelerometer had changed during the three months' period of testing.

A study of the data obtained shows that the individual tests are to be regarded as very satisfactory. A plotting of any of the tabulated impact reactions against corresponding heights of drop will give smooth, consistent curves.

A general idea of the scope of the experiment may be had by referring to Figure 2, which shows the characteristic curve for each type tested. This diagram is much too confusing to be of use in drawing conclusions. It was considerably simplified by averaging into one curve all values for similar types whose maximum reactions lay within a zone 1,000 pounds

TABLE 2.—*Tabulation of impact reactions expressed in thousand pounds*

TYPE 1.—2-INCH TOPEKA; 6-INCH 1:3:7 CONCRETE BASE, CONNECTICUT AVENUE, MARYLAND

Height of drop (inches)	Test No.												Average for type
	1	2	3	4	5	6	7	8	9	10	11	12	
0.33	11.7	11.9	11.0	10.7	11.0	12.4	11.3	12.1	12.0	11.4	12.3	11.7	11.6
.66	16.6	18.1	17.7	15.8	15.8	18.9	18.2	18.7	18.1	18.0	18.2	17.5	17.6
1.00	23.2	24.1	23.5	22.9	23.8	25.3	23.5	24.9	24.3	23.5	24.3	23.5	23.9
1.33	28.8	28.9	30.3	27.1	26.7	31.7	30.0	30.6	30.8	30.2	29.9	26.8	29.3
1.66	33.3	34.7	35.3	31.8	32.4	37.2	35.2	36.1	35.9	35.8	35.8	33.2	34.7

TYPE 2.—2-INCH BITUMINOUS CONCRETE (DISTRICT OF COLUMBIA SPECIFICATION); 6-INCH 1:3:7 CONCRETE BASE, CONNECTICUT AVENUE, MARYLAND

0.33	10.2	11.3	11.4	12.5	12.4	11.5	10.1	11.0	11.5	11.8	12.3	11.6	11.5
.66	16.2	18.3	19.0	18.7	18.2	16.8	15.3	16.4	19.1	18.7	18.8	18.0	17.8
1.00	22.6	24.0	25.1	25.4	24.7	23.1	21.4	21.6	25.3	24.6	25.1	24.9	24.0
1.33	29.4	30.2	32.1	32.1	30.4	29.7	27.0	25.7	31.5	30.7	31.5	32.1	30.2
1.66	33.7	35.6	37.1	37.6	35.9	34.3	31.6	31.2	37.5	36.2	37.1	37.8	35.5

TYPE 3.—3-INCH COMBINED SHEET ASPHALT AND BINDER; 6-INCH 1:3:7 CONCRETE BASE, CONNECTICUT AVENUE, DISTRICT OF COLUMBIA

0.33	11.7	11.9	13.0	12.5	12.6	11.1	-----	-----	-----	-----	-----	-----	12.1
.66	18.7	18.8	18.9	20.9	19.5	16.9	-----	-----	-----	-----	-----	-----	19.0
1.00	25.1	25.8	26.1	27.0	25.7	22.9	-----	-----	-----	-----	-----	-----	25.4
1.33	32.2	31.8	31.6	34.1	32.1	29.1	-----	-----	-----	-----	-----	-----	31.8
1.66	38.9	38.1	38.1	40.7	38.5	34.8	-----	-----	-----	-----	-----	-----	38.2

TYPE 4.—3-INCH COMBINED SHEET ASPHALT AND BINDER; 7-INCH 1:3:6 CONCRETE BASE, CONNECTICUT AVENUE, DISTRICT OF COLUMBIA

0.33	12.8	13.2	13.0	12.6	12.9	13.2	-----	-----	-----	-----	-----	-----	13.0
.66	19.8	20.6	20.0	19.2	20.6	20.4	-----	-----	-----	-----	-----	-----	20.1
1.00	27.0	27.4	26.5	26.7	27.9	27.8	-----	-----	-----	-----	-----	-----	27.2
1.33	33.7	33.7	33.6	34.0	34.4	35.0	-----	-----	-----	-----	-----	-----	34.1
1.66	40.5	40.1	40.1	40.3	41.1	42.3	-----	-----	-----	-----	-----	-----	40.7

TYPE 5.—1-INCH SHEET ASPHALT ON SURFACE-TREATED, WATER-BOUND MACADAM BASE, CHEVY CHASE CIRCLE, DISTRICT OF COLUMBIA

0.33	9.2	9.6	10.1	-----	-----	-----	-----	-----	-----	-----	-----	-----	9.6
.66	14.1	12.8	14.9	-----	-----	-----	-----	-----	-----	-----	-----	-----	13.9
1.00	20.7	17.5	19.4	-----	-----	-----	-----	-----	-----	-----	-----	-----	19.2
1.33	24.3	21.7	25.4	-----	-----	-----	-----	-----	-----	-----	-----	-----	23.8
1.66	28.2	26.9	29.8	-----	-----	-----	-----	-----	-----	-----	-----	-----	28.3

TYPE 6.—SURFACE-TREATED, WATERBOUND MACADAM, GEORGETOWN-ALEXANDRIA ROAD, ARLINGTON COUNTY, VA.

0.33	11.4	11.1	12.0	-----	-----	-----	-----	-----	-----	-----	-----	-----	11.8
.66	16.7	18.0	17.6	-----	-----	-----	-----	-----	-----	-----	-----	-----	17.4
1.00	21.9	22.9	23.0	-----	-----	-----	-----	-----	-----	-----	-----	-----	22.6
1.33	26.6	28.9	29.2	-----	-----	-----	-----	-----	-----	-----	-----	-----	28.2
1.66	31.7	33.6	34.4	-----	-----	-----	-----	-----	-----	-----	-----	-----	33.2

TABLE 2.—*Tabulation of impact reactions expressed in thousand pounds*—Continued

TYPE 7.—6-INCH 1:2:4 CONCRETE

Height of drop (inches)	Test No.												Average for type
	1	2	3	4	5	6	7	8	9	10	11	12	
	Woodbine Street, Maryland			Williams Lane, Maryland			Underwood Street, Maryland		Thornapple Street, Maryland				
0.33	11.8	9.2	11.3	11.1	10.2	11.8	11.4	11.3	10.8	10.8	11.9	-----	-----
.66	17.6	17.0	17.2	18.1	17.2	18.0	17.9	17.9	18.4	18.0	19.2	-----	-----
1.00	23.8	22.7	24.4	25.0	24.8	24.7	24.8	23.9	24.2	24.8	24.6	-----	-----
1.33	30.1	28.7	30.3	31.7	32.1	31.6	31.5	30.1	29.4	32.0	33.1	-----	-----
1.66	36.4	34.9	37.3	38.1	38.4	38.3	37.5	35.5	35.8	39.1	38.5	-----	-----
	Northampton Street, District of Columbia		Chevy Chase Parkway, District of Columbia		Nebraska Avenue, District of Columbia								
0.33	12.9	12.4	13.1	13.6	13.2	13.6	-----	-----	-----	-----	-----	-----	11.8
.66	20.3	19.8	19.8	20.1	19.4	20.7	-----	-----	-----	-----	-----	-----	18.7
1.00	26.5	26.3	26.1	26.6	27.0	26.8	-----	-----	-----	-----	-----	-----	25.1
1.33	33.3	33.3	33.7	33.6	32.4	32.7	-----	-----	-----	-----	-----	-----	31.7
1.66	38.7	38.6	39.4	39.8	38.8	39.4	-----	-----	-----	-----	-----	-----	38.0

TYPE 8.—6-INCH 1:2:3 CONCRETE

	Columbia Pike, Virginia			Test road, Arlington Experiment Station, Virginia									
0.33	12.0	12.8	12.2	11.8	12.9	13.1	13.7	12.4	12.4	-----	-----	-----	12.6
.66	19.7	19.5	18.8	18.7	19.6	19.4	20.3	20.8	20.7	-----	-----	-----	19.7
1.00	25.8	26.7	26.8	26.4	26.7	25.6	27.0	28.1	27.8	-----	-----	-----	26.8
1.33	34.2	34.3	35.2	32.1	33.1	31.1	34.4	35.6	34.8	-----	-----	-----	33.9
1.66	40.6	41.8	42.1	39.9	39.1	38.5	40.8	41.6	40.8	-----	-----	-----	40.6

TYPE 9.—7-INCH 1:2:4 CONCRETE, CONNECTICUT AVENUE, DISTRICT OF COLUMBIA

0.33	14.3	13.2	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	13.7
.66	20.3	20.7	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	20.5
1.00	27.3	27.0	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	27.1
1.33	33.2	35.6	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	34.4
1.66	39.9	40.8	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	40.3

TYPE 10.—7-INCH 1:2:3 CONCRETE, COLUMBIA PIKE, VIRGINIA

0.33	12.8	13.1	12.2	11.8	-----	-----	-----	-----	-----	-----	-----	-----	12.5
.66	19.8	18.9	19.1	19.4	-----	-----	-----	-----	-----	-----	-----	-----	19.3
1.00	25.9	27.3	26.7	26.5	-----	-----	-----	-----	-----	-----	-----	-----	26.6
1.33	32.9	35.3	35.0	33.9	-----	-----	-----	-----	-----	-----	-----	-----	34.3
1.66	41.5	41.1	41.1	40.8	-----	-----	-----	-----	-----	-----	-----	-----	41.1

TYPE 11.—8-INCH 1:2:3 CONCRETE

	Columbia Pike, Virginia				Test road, Arlington Experiment Station, Virginia								
0.33	11.9	12.7	12.1	13.3	12.8	13.3	14.1	13.1	14.2	12.9	-----	-----	13.0
.66	20.0	19.0	20.4	20.0	19.6	19.5	20.8	19.4	19.9	20.5	-----	-----	19.9
1.00	27.6	26.7	27.9	27.5	27.3	26.9	25.3	25.7	27.4	26.9	-----	-----	26.9
1.33	35.3	34.1	33.7	35.1	33.6	33.7	32.4	32.4	35.6	34.1	-----	-----	34.0
1.66	41.6	42.9	41.0	42.0	39.1	39.8	38.6	38.7	41.2	40.3	-----	-----	40.5

TYPE 12.—10-INCH 1:2:3 CONCRETE, TEST ROAD, ARLINGTON EXPERIMENT STATION, VIRGINIA

0.33	10.2	13.2	12.9	13.0	12.9	13.1	-----	-----	-----	-----	-----	-----	12.5
.66	18.9	20.4	19.6	20.4	19.4	20.5	-----	-----	-----	-----	-----	-----	19.9
1.00	26.8	27.3	27.5	27.2	27.8	28.0	-----	-----	-----	-----	-----	-----	27.4
1.33	32.7	34.3	33.8	33.8	34.8	35.6	-----	-----	-----	-----	-----	-----	34.2
1.66	40.8	40.5	41.5	40.8	42.5	42.2	-----	-----	-----	-----	-----	-----	41.4

TABLE 2.—*Tabulation of impact reactions expressed in thousand pounds*—Continued

TYPE 13.—2½-BRICK ON ¾-INCH SAND CUSHION; 6-INCH 1:1½:3: CONCRETE BASE, CIRCULAR TEST TRACK, ARLINGTON EXPERIMENT STATION, VIRGINIA

Height of drop (inches)	Test No. 1	2	3	4	5	6	7	8	9	10	11	12	Average for type
0.33	11.2	12.9	12.0	11.6									11.9
.66	18.3	19.5	19.4	19.5									19.2
1.00	26.9	26.9	27.0	26.2									26.7
1.33	31.9	34.4	31.0	33.3									32.6
1.66	39.3	41.1	40.1	40.1									40.1

TYPE 14.—2½-INCH BRICK ON ¾-INCH 1:4 SAND-CEMENT CUSHION; 6-INCH 1:1½:3 CONCRETE BASE, CIRCULAR TEST TRACK, ARLINGTON EXPERIMENT STATION, VIRGINIA

0.33	12.0	12.9	11.9										12.3
.66	19.9	20.9	19.4										20.1
1.00	26.2	27.1	26.1										26.5
1.33	33.7	34.8	33.0										33.8
1.66	40.5	41.2	40.2										40.6

TYPE 15.—4-INCH BRICK ON ¾-INCH SAND CUSHION; 6-INCH 1:1½:3 CONCRETE BASE, CIRCULAR TEST TRACK, ARLINGTON EXPERIMENT STATION, VIRGINIA

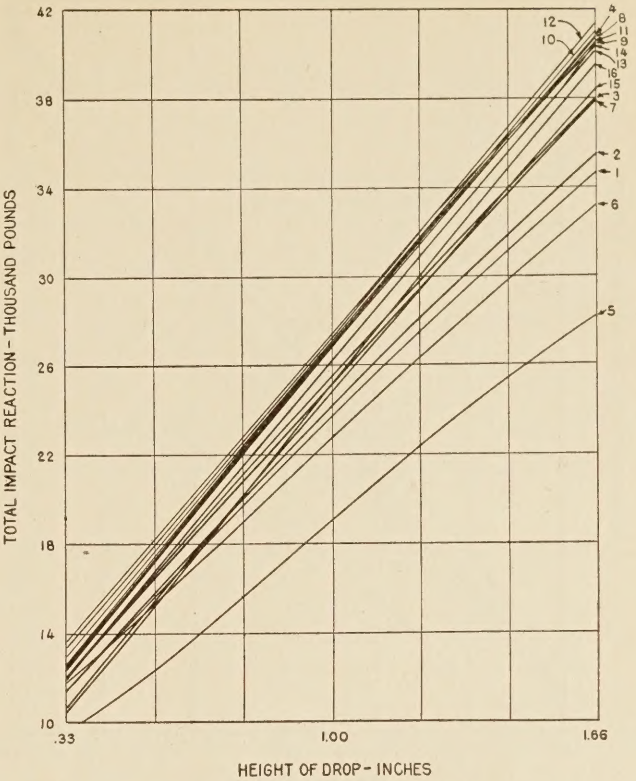
0.33	10.9	11.1	8.8										9.9
.66	17.7	19.1	16.5										17.8
1.00	24.9	26.0	23.4										24.8
1.33	31.2	32.5	30.7										31.5
1.66	38.2	39.5	38.1										38.6

TYPE 16.—4-INCH BRICK ON ¾-INCH 1:4 SAND-CEMENT CUSHION 6-INCH 1:1½:3 CONCRETE BASE, CIRCULAR TEST TRACK, ARLINGTON EXPERIMENT STATION, VIRGINIA

0.33	9.2	10.9	11.8										10.6
.66	16.4	18.1	18.6										17.7
1.00	23.4	25.3	26.6										25.1
1.33	30.7	33.1	34.0										32.6
1.66	38.1	40.1	40.6										39.6

wide (in the case of brick sections this zone was 2,000 pounds wide), the values for each height of drop being averaged and the average curve drawn through these points. The simplified curves are shown in Figure 3 and will be used later in making certain comparisons.

In order to still further clarify the presentation of the data, special limited groupings of types directly comparable are made for purposes of study. These will be found in Figures 4, 5, 8, 9, and 13.



1.—2-inch Topeka; 6-inch 1 : 3 : 7 concrete base.
2.—2-inch bituminous concrete, District of Columbia specification; 6-inch 1 : 3 : 7 concrete base.
3.—3-inch combined sheet asphalt and binder; 6-inch 1 : 3 : 7 concrete base.
4.—3-inch combined sheet asphalt and binder; 7-inch 1 : 3 : 6 concrete base.
5.—1-inch sheet asphalt; surface-treated water bound macadam base.
6.—Surface-treated water bound macadam.
7.—6-inch 1 : 2 : 4 concrete.
8.—6-inch 1 : 2 : 3 concrete.
9.—7-inch 1 : 2 : 4 concrete.
10.—7-inch 1 : 2 : 3 concrete.
11.—8-inch 1 : 2 : 3 concrete.
12.—10-inch 1 : 2 : 3 concrete.
13.—2½-inch brick; ¾-inch sand cushion; 6-inch 1 : 1½ : 3 concrete base.
14.—2½-inch brick; ¾-inch sand-cement cushion; 6-inch 1 : 1½ : 3 concrete base.
15.—4-inch brick; ¾-inch sand cushion; 6-inch 1 : 1½ : 3 concrete base.
16.—4-inch brick; ¾-inch sand-cement cushion; 6-inch 1 : 1½ : 3 concrete base.

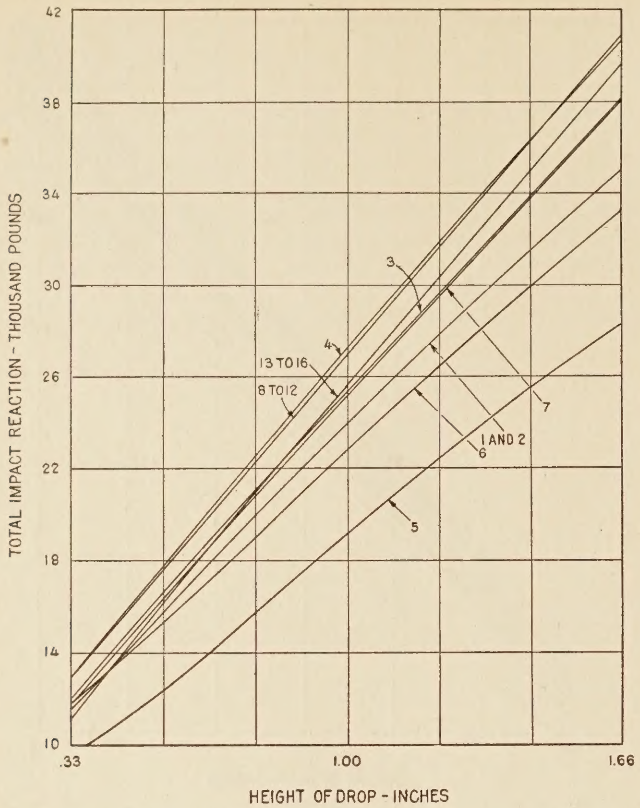
FIG. 2.—CHARACTERISTIC CURVES FOR EACH TYPE OF PAVEMENT TESTED

BITUMINOUS PAVEMENTS ON NONRIGID BASES SHOW CUSHIONING PROPERTIES

In considering the data, certain questions naturally arose, and these have been set forth below, the pertinent data discussed, and, wherever possible, conclusions drawn.

1. Are bituminous-surfaced types (sheet asphalt, Topeka, bituminous concrete, etc., laid upon rigid or nonrigid bases) capable of cushioning or absorbing the shock of impact forces? Referring to either Figure 2 or 3, it is indicated that surface-treated and partially penetrated water-bound macadam bases are distinctly cushioning to motor-truck impact, since for identical impact conditions these types offer reactions which are appreciably lower than those obtained for the rigid types.

A marked tendency for the pavement to deflect over a considerable area and then spring back was observed in the case of both of the nonrigid types tested. This deflection amounted to at least one-half inch directly under the impact wheel and was noticeable even at a distance of 2 or 3 feet from the center of impact. The deflected area seemed to resume its original position after removal of the load except at the point where the impact wheel had struck; here it was permanently deformed. In the case of the rigid types, however, the deflections were obviously very small, so small, indeed, as not to be apparent to the eye, although they could be felt. The lowering of impact reactions in the case of the nonrigid types is attributed to this characteristic, which leaves the observer with the same impression he acquired when he had learned to ease the shock of a hard-thrown ball by moving his hands in the direction



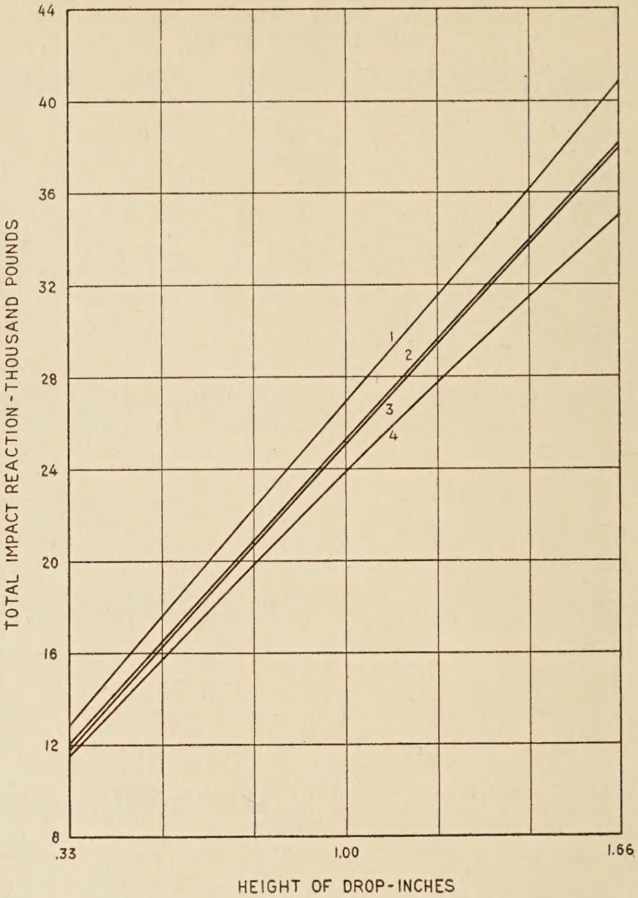
1 and 2.—2-inch Topeka and 2-inch bituminous concrete; 6-inch 1 : 3 : 7 concrete base.
3.—3-inch combined sheet asphalt and binder; 6-inch 1 : 3 : 7 concrete base.
4.—3-inch combined sheet asphalt and binder; 7-inch 1 : 3 : 6 concrete base.
5.—1-inch sheet asphalt; surface-treated water-bound macadam base.
6.—Surface-treated water-bound macadam.
7.—6-inch 1 : 2 : 4 concrete.
8 to 12.—6, 7, 8, and 10 inch 1 : 2 : 3 concrete; 7-inch 1 : 2 : 4 concrete.
13 to 16.—2½ and 4 inch brick on sand and sand-cement cushion; 6-inch 1 : 1½ : 3 concrete base.

Fig. 3.—Simplified Characteristic Curves Derived by Grouping into One Curve All Curves, for Similar Types, Whose Maximum Reactions Lay Within a Zone 1,000 Pounds Wide, Except for Brick Sections Where the Zone was 2,000 Pounds Wide

of the ball's motion at the instant of impact rather than to depend entirely upon the cushioning properties of the padding in his glove—the impression that cushioning is dependent upon relatively large deflections. Attention was called to this in a previous article in Public Roads.[4]

Barring the flexible types which have just been discussed and considering only the rigid types of bituminous-surfaced pavements, it is not as easy to arrive at definite conclusions as it was in the former case. Referring to Figure 3, it will be seen that 2-inch Topeka and bituminous concrete laid on 6-inch 1:3:7 concrete base, 3-inch sheet asphalt and binder on 6-inch 1:3:7 concrete base, and 3-inch sheet asphalt and binder on 7-inch 1:3:6 concrete base, occupy lower, middle, and upper positions, respectively, of the zone covered by the rigid types. Figures 4 and 5 also show these curves compared with closely comparable types.

[4] Teller, L. W., Impact Tests on Concrete Pavement Slabs, Public Roads, vol. 5, No. 2, April, 1924.



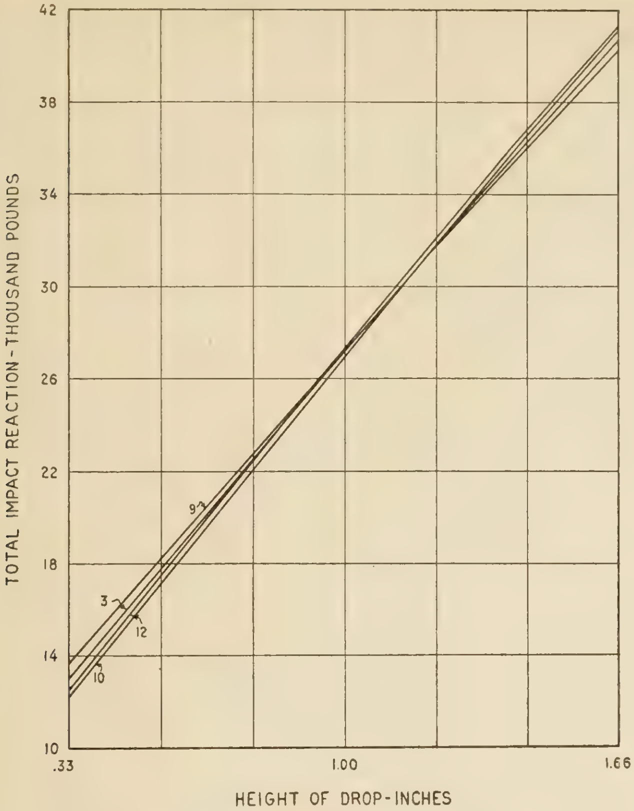
1.—6, 8, and 10 inch 1 : 2: 3 concrete and 7-inch 1 : 2: 4 concrete.
2.—2-inch combined sheet asphalt and binder; 6-inch 1 : 3 : 7 concrete base.
3.—6-inch 1 : 2 : 4 concrete.
4.—2-inch Topeka and 2-inch bituminous concrete; 6-inch 1 : 3 : 7 concrete base.

Fig. 4.—Limited Grouping of Curves for Comparative Purposes

Whether the reduction in reaction of approximately 10 per cent indicated in Figure 3 for the 2-inch bituminous top on 6-inch 1:3:7 base, as compared with the uncovered concrete pavement, is really due to cushioning properties of the top is open to considerable doubt. In the first place, we have no way of knowing whether or not the concrete base was badly cracked at or near the point of test. This pavement, Connecticut Avenue, Chevy Chase, Md., was laid by the Bureau of Public Roads in 1912 for experimental purposes. In it are concrete sections without tops laid at the same time as the bituminous-surfaced sections under discussion and of the same thickness, namely, 6 inches. These have no top, but are of a richer mix (1:1¾:3) and have been subjected to the same traffic. These concrete sections are badly cracked, and it is therefore reasonable to suppose that the concrete base under the 2-inch top is likewise badly cracked. Such a condition would, of course, have the effect of lowering the reactions.[5]

Similar doubt clouds the small reduction shown by the 3-inch sheet asphalt and binder laid on 6-inch 1:3:7 base as compared with the uncovered concrete

[5] Since these tests were completed, in the course of another investigation, several cores were drilled from the base of the section just described, and it was found that the base is badly cracked, although in general the alignment of the base surface is well preserved.



3.—3-inch combined sheet asphalt and binder; 7-inch 1 : 3 : 6 concrete base.
9.—7-inch 1 : 2 : 4 concrete.
10.—7-inch 1 : 2 : 3 concrete.
12.—10-inch 1 : 2 : 3 concrete.

Fig. 5.—Limited Grouping of Curves for Comparative Purposes

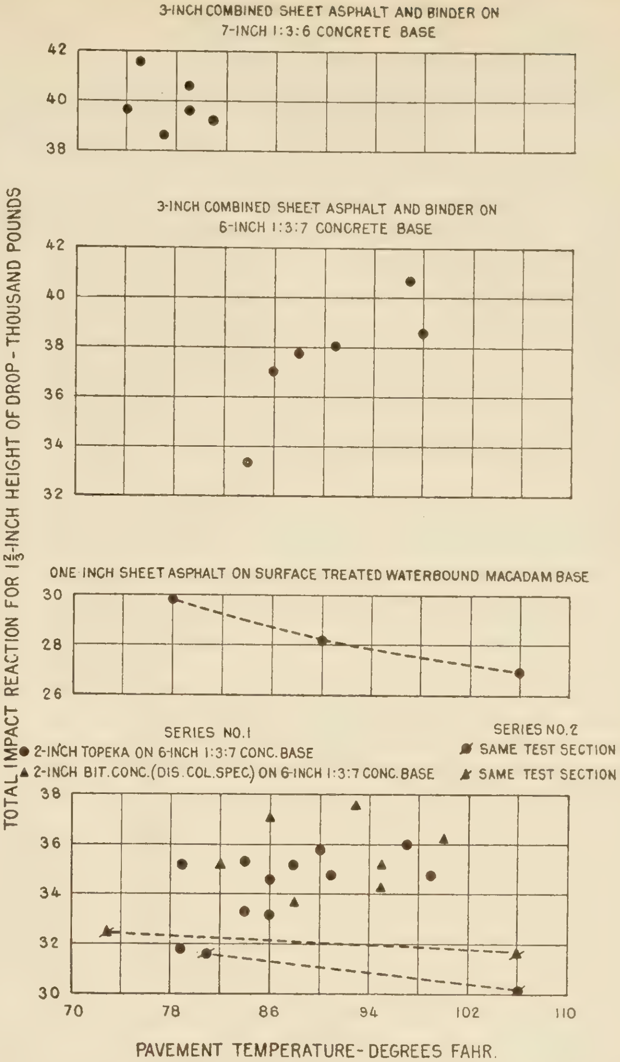


Fig. 6.—Curves Showing Influence of Pavement Temperature upon Impact Reaction

pavement. This pavement was laid in 1920 and shows from numerous surface cracks and sunken spots evidence of base cracking, although the test sections were chosen at places where the pavement showed a minimum of such symptoms.

The 3-inch sheet asphalt and binder on 7-inch 1:3:6 base (fig. 5) was, fortunately, a new pavement, completed about two weeks before it was tested. It shows no reduction in reaction whatever as compared with a 7-inch 1:2:4 uncovered slab, for example, which strengthens the belief that possibly base cracking was responsible for the reductions discussed above.

Due to the impracticability of observing the condidition of the bases mentioned above, the foregoing explanations are admittedly not infallible.

In view of the foregoing it might be said that the data support the conclusion that while the bituminous-surfaced pavements show, in general, somewhat lower reactions for the same impact conditions than do the unsurfaced concrete pavements, these differences are not consistent and are of no greater magnitude than the differences found between the various unsurfaced concrete pavements tested.

TEMPERATURE EFFECTS CONSIDERED

2. Do the impact reactions offered by bituminous-surfaced pavements vary with changes in pavement temperature? It is reasonable to believe that the behavior of bituminous-surfaced pavements would be affected by temperature, and the data were analyzed with this in mind. Referring to Figure 6, it will be seen that for types employing a bituminous top upon a rigid base no consistent or marked change occurs, and the question must be answered in the negative for the types and range of temperatures studied. Limited data in the case of one nonrigid type (1-inch sheet asphalt on surface-treated water-bound macadam) does, however, show a consistent indication of decrease in reaction with increased temperature, the impact reaction being 10 per cent higher at a pavement temperature of 78° F. than at 106°.

3. Do the permanent deformations caused by successive impact blows upon a bituminous-surfaced pavement increase as temperatures increase? By referring to Figure 7 it will be seen that for the types and temperatures studied there is evidence of a very small increase in permanent deformations for certain types employing a rigid base, but the evidence is neither marked nor consistent. In the case of one nonrigid type (1-inch sheet asphalt on surface-treated water-bound macadam base) there is, however, a marked increase, the deformation at 106° F. being four times as great as at 78°.

4. Do the reactions of a given bituminous-surfaced pavement on rigid base increase with additional compacting, such, for example, as would be occasioned by

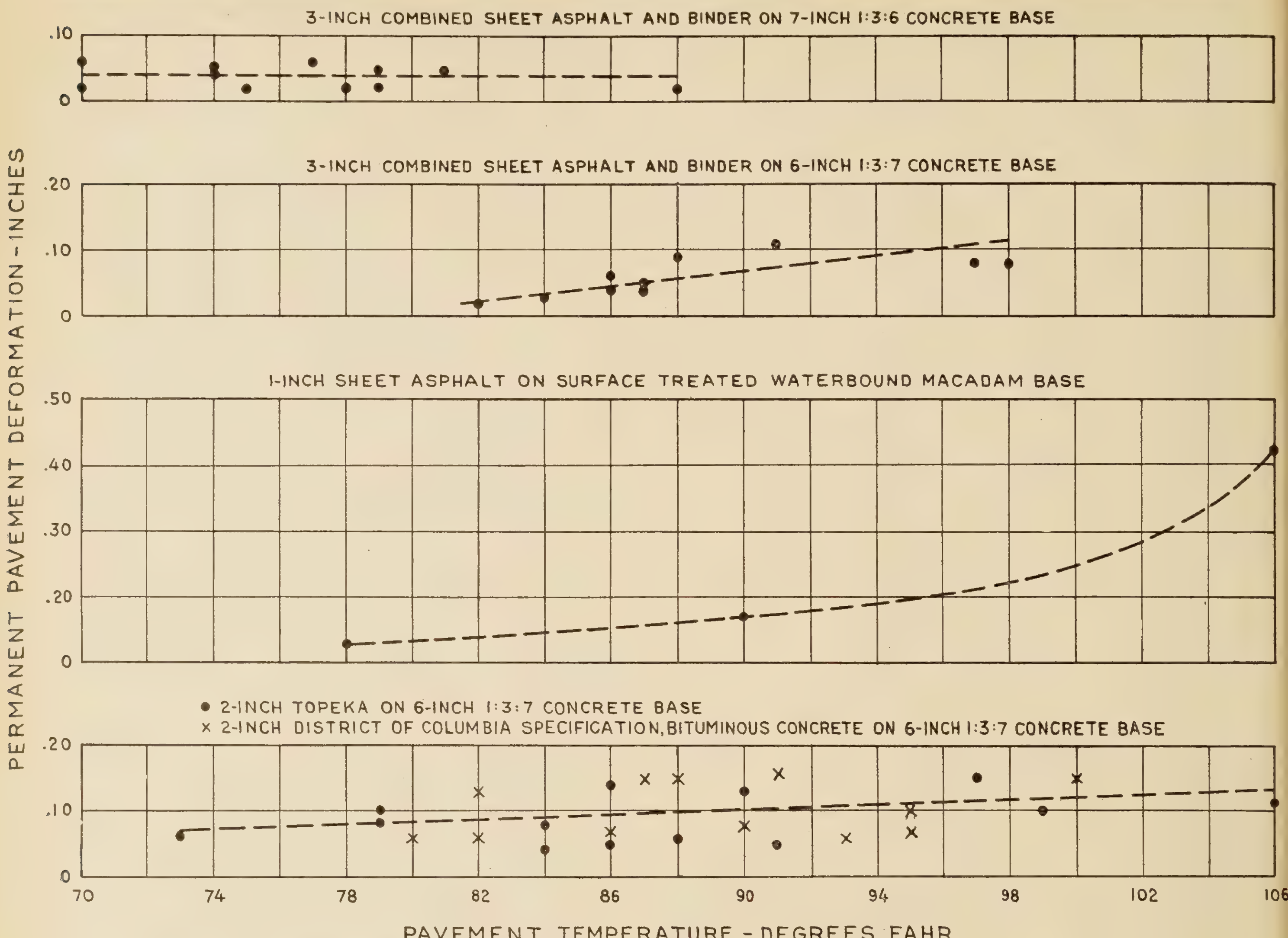


Fig. 7.—Curves Showing Influence of Pavement Temperature upon Pavement Deformation

repeated impact blows upon the same spot? On several occasions after the necessary 15 blows of increasing intensity had been struck, the machine was moved about a foot and blows struck from the maximum height of drop only. The reactions offered by the second spot differed in no essential from those corresponding to maximum height of drop for the first spot. The question must, therefore, be answered in the negative.

RESULTS ON UNCOVERED CONCRETE DISCUSSED

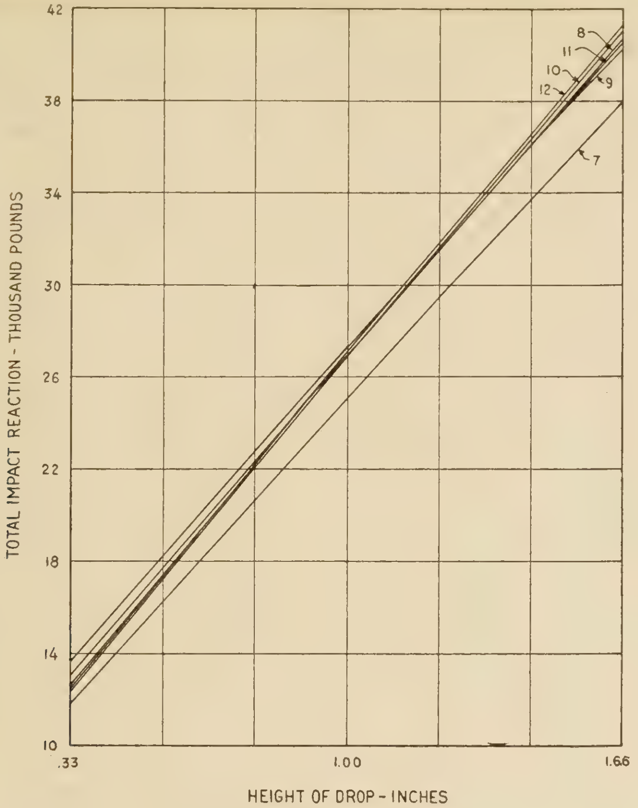
5. Considering uncovered concrete sections as a class, what effect, if any, does the thickness have upon impact reactions? As the difference in deflection of 6, 8, and 10 inch pavement slabs under a given load is relatively small, one would not expect material variations in reaction for concrete pavements within this range, and the data indicate this to be true. Referring to Figure 8, it will be noticed that all save one of the characteristic curves of uncovered concrete types are bunched, the only one showing a significant difference being the 6-inch 1:2:4 type. The fact that this one type does not line up with the others leads one to the belief that one can expect as great differences to exist between the uncovered types as is to be found when comparing the covered and uncovered types.

BRICK PAVEMENTS SHOW ONLY SLIGHT CUSHIONING

6. Do brick pavements, consisting of a concrete base, either a sand or sand-cement cushion, and a vitrified brick wearing surface exhibit any cushioning or shock-absorbing properties? In Figure 3 it will be seen that the characteristic curve for such types occupies a position corresponding closely to that of the average of all uncovered concrete sections. It seems, therefore, that this type of pavement does not exhibit cushioning properties to any significant extent.

Referring to Figure 9, it will be seen that, for a given thickness of brick, sand and sand-cement cushions yield about the same reactions, but that there is some indication that the 2½-inch brick sections give rise to somewhat higher impact reactions that do the 4-inch sections. The difference is small, however, and is not thought to be particularly significant in view of the limited number of tests made.

The fact that there appears to be little difference in the reactions for brick surfaces laid on the two types of bedding material is, however, of considerable interest in view of the recent accelerated traffic tests on the same sections, in which it was found that there was about twice as much breakage where bricks rested on



7.—6-inch 1 : 2 : 4 concrete.
8.—6-inch 1 : 2 : 3 concrete.
9.—7-inch 1 : 2 : 4 concrete.
10.—7-inch 1 : 2 : 3 concrete.
11.—8-inch 1 : 2 : 3 concrete.
12.—10-inch 1 : 2 : 3 concrete.

Fig. 8.—Limited Grouping of Curves for Comparative Purposes

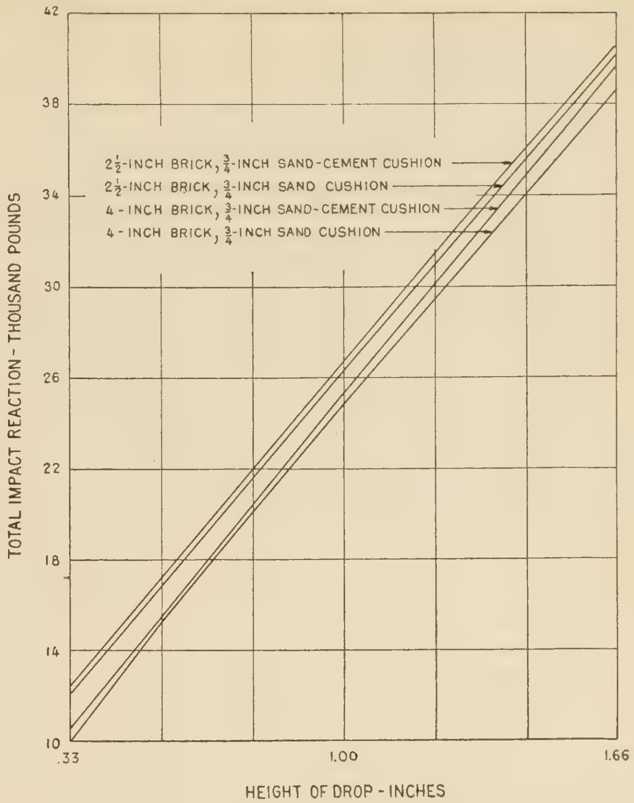


Fig. 9.—Limited Grouping of Curves for Brick Sections for Comparative Purposes. All Brick Laid Upon 6-inch 1 : 1½ : 3 Concrete

sand-cement cushions as when they rested upon sand cushions.[6]

SUMMARY OF CONCLUSIONS

1. For the limited data in hand it is indicated that bituminous pavements of the nonrigid type, such as surface-treated waterbound macadam, may substantially cushion the effect of impact forces.

2. Bituminous-surfaced pavements, such as sheet asphalt, Topeka, etc., laid upon concrete bases show, in general, some indications of cushioning impact forces, but the magnitude of this cushioning effect appears to be relatively small, and in some cases there is considerable doubt as to its actual existence. It should also be pointed out that the differences observed between reactions for the bituminous-surfaced pavements and those for the unsurfaced ones are no greater than the differences found to exist between reactions for the various sections of unsurfaced concrete pavement.

3. In general, the impact reactions of bituminous-surfaced pavements on rigid bases fail to show any marked or consistent change with changes in pavement temperatures up to 106° F. However, in the case of one nonrigid type of bituminous pavement (1-inch sheet asphalt on waterbound macadam base) a consistent substantial decrease in reaction with increased temperatures was noted.

4. The data regarding the permanent deformation due to successive impact forces upon bituminous-surfaced pavements on rigid bases show only a small and not very consistent tendency to increase with increasing temperatures up to 106° F. The limited tests of nonrigid types, however, show a marked tendency for these types to suffer greater permanent deformations as the pavement temperatures increase.

5. The impact reactions of bituminous-surfaced pavements on rigid bases show no tendency to increase as the section becomes more compact, due to repeated impact blows.

6. Within the range of thicknesses studied, impact reactions of uncovered concrete pavements do not appear to be affected materially or consistently by variations of slab thickness.

7. Brick types in which the brick wearing surface was bituminous filled and rested upon a sand or sand-cement bedding course on a concrete base show no marked tendency to cushion impact forces.

8. The reactions of brick types employing plain sand bedding courses are practically the same as those of types employing sand-cement bedding.

RESULTS WITH IMPACT MACHINE CONFIRMED BY TESTS WITH MOTOR TRUCK EQUIPPED WITH ACCELEROMETER

It was thought advisable to conduct the main tests with the impact machine and to supplement them with tests using actual motor-truck impacts. It was recognized that impacts could be much more closely controlled with the impact machine, but data obtained with the truck were suggested as a means of making sure that the factors affecting the impact-machine reactions would also affect actual motor-truck impacts.

[6] Pauls, J. T., and Teller, L. W., Thin Brick Pavements Studied, Public Roads, vol. 7, No. 7, September, 1926.

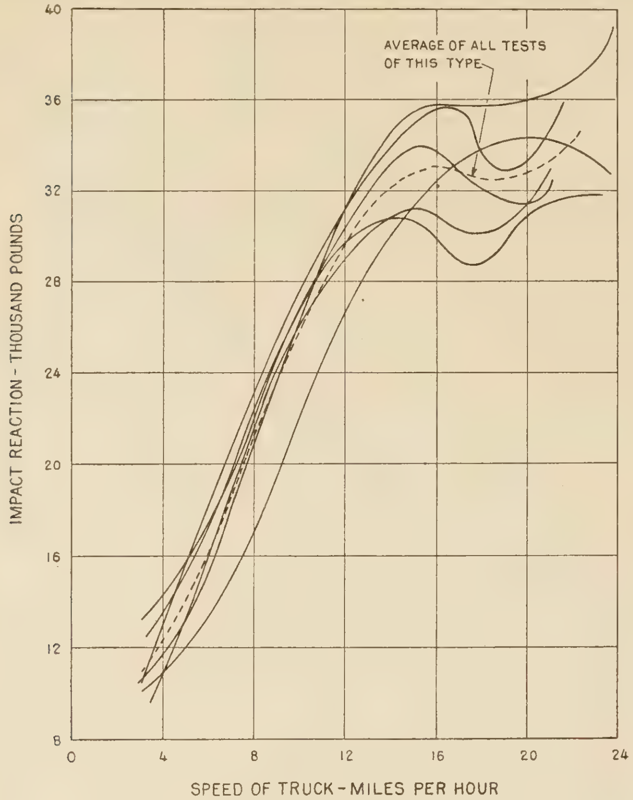


FIG. 10.—MOTOR-TRUCK IMPACTS ON A 2-INCH TOPEKA SURFACE ON A 6-INCH 1 : 3 : 7 BASE

FIG. 12.—MOTOR-TRUCK IMPACTS ON A 6-INCH 1 : 2 : 4 CONCRETE

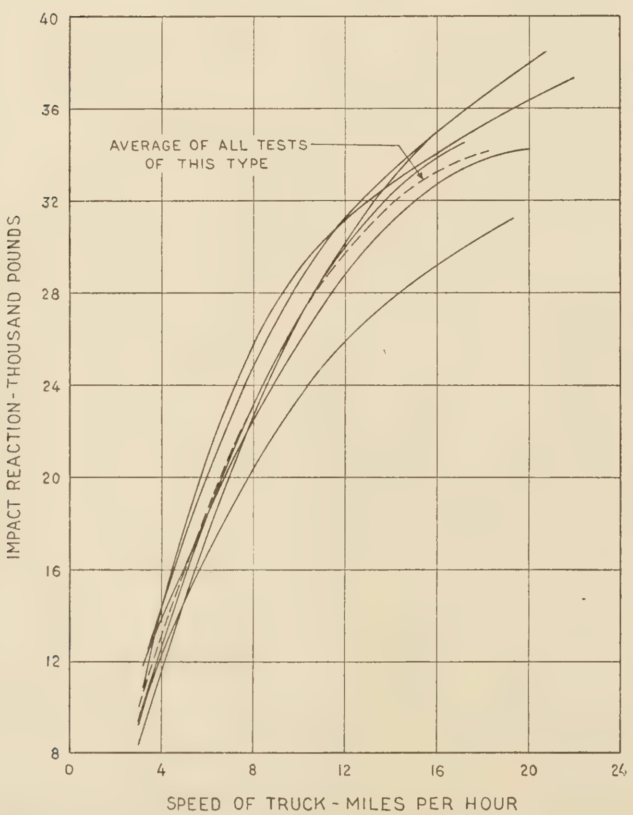


FIG. 11.—MOTOR-TRUCK IMPACTS ON A 2-INCH BITUMINOUS CONCRETE ON A 6-INCH 1 : 3 : 7 BASE

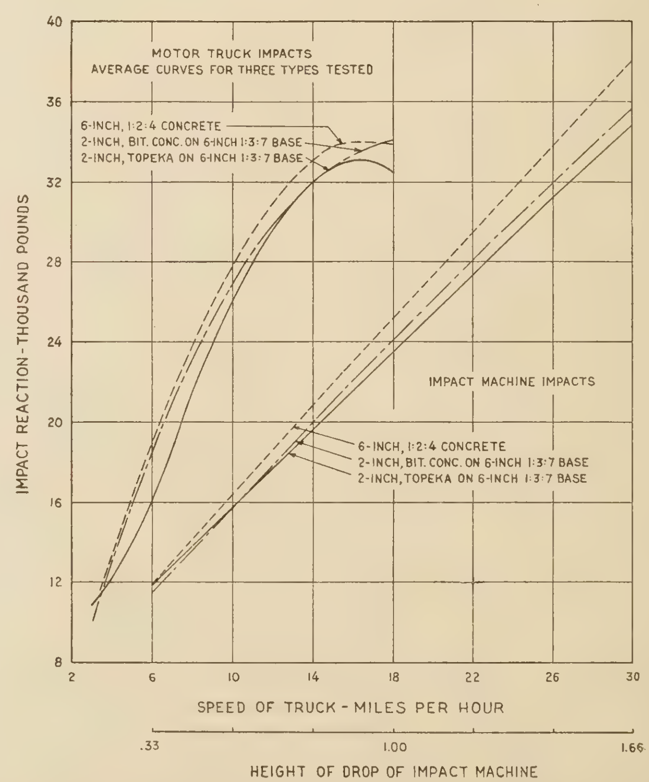


FIG. 13.—LIMITED GROUPING SHOWING MOTOR-TRUCK AND IMPACT-MACHINE REACTIONS FOR THE THREE TYPES FOR WHICH ACTUAL MOTOR-TRUCK IMPACTS ARE AVAILABLE

Accordingly, when sufficient data had been secured with the impact machine to indicate that some differences existed between the characteristic curves for the first three types tested, namely, 2-inch Topeka on 6-inch 1:3:7 base, 2-inch bituminous concrete on 6-inch 1:3:7 base, and 6-inch 1:2:4 concrete (curves 1, 2, and 7, fig. 2), it was thought desirable to see if similar characteristic curves, developed by actual motor-truck impacts, would show a tendency to differ in the same general direction. Accordingly, a 2-ton truck with rear tires comparable to the tire used on the impact-machine plunger was run over a $\frac{9}{16}$-inch inclined obstruction in such a manner that one rear wheel would strike upon the pavement at a section[7] previously tested by the impact machine.

The force of this impact was measured by means of an accelerometer[8] essentially the same as that used in the impact-machine tests, the instrument being mounted on one of the rear wheels. The truck was so loaded that impacts delivered by it at varying speeds would about cover the range of impacts secured with the impact machine.

The speed of the truck was determined by means of stop-watch measurements and checked and controlled be speedometer observations.

The data obtained have been plotted in Figures 10, 11, and 12 for the three types tested and the average results for each of these types plotted separately in Figure 13, upon which have also been plotted the impact-machine reactions for these types for comparative purposes.

Truck Equipped with Accelerometer. The Rear Wheel is About to Drop from the Wedge-Shaped Obstruction

From Figure 13 it will be seen that the motor-truck impact reaction curves show a definite tendency to differ in the same general direction as do the characteristic curves obtained with the impact machine and by about the same general amounts.

[7] The impact-machine wheel was made to strike 3 feet 4 inches from the pavement edge, the truck wheel 8 feet from the same edge. This was necessary on account of traffic conditions.

[8] For a more detailed description of this apparatus see Public Roads, vol. 7, No. 4, Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness, by James A. Buchanan and J. W. Reid.

TRUCK IS A BIG FACTOR IN FRUIT TRANSPORT[1]

The marked increase in the use of motor trucks for hauling farm produce direct from farms to markets is shown in a survey in New York City which brought out that from 20 to 30 per cent of the supply of leading fruits on the New York market is hauled into the city by motor truck.

For about three months in midseason New York gets nearly one-third its peach supply, one-fourth its tomatoes, and one-fifth its apples, by motor truck, and sometimes during the busy season more than one-half the New Jersey produce supply moves in trucks. The survey was made by the Bureau of Agricultural Economics of the United States Department of Agriculture and the New York Food Marketing Research Council cooperating.

The tendency to change from the horse-drawn wagon or the railroad car to the motor truck has been going on for a dozen years. The New York dealers wanted to know just how much produce was coming by truck and about 45 of them agreed to give by telephone each morning the number of packages of each product received by truck, according to the State of origin.

The investigation was limited to five important lines of produce—peaches, cantaloupes, tomatoes, apples, and peppers. The reporting started July 20, and continued through the peak period to October 22. Information gathered in this way does not give the complete total of receipts coming by motor truck into the New York area. Produce arrives by this means at the Newark, Harlem, Gansevoort, and Wallabout markets, as well as the several farmers' markets, but it appears that the bulk of New Jersey motor-truck shipments distributed in New York City has been accounted for.

During the period of investigation the five products reported were shipped by motor-truck to New York from points as distant as Virginia, Maryland, and Delaware, as well as from the near-by sections of New Jersey, Pennsylvania, Long Island (N. Y.), and Connecticut. Excepting cantaloupes from Virginia, Maryland, and Delaware, the New Jersey motor-truck receipts so far exceeded those from other States that the latter appeared insignificant.

Thirty per cent of all peaches arriving at New York during the period reported came by motor truck. Twenty-five per cent of the tomatoes, 20 per cent of the apples, and 9 per cent of the peppers also arrived in this way.

Taking a single week during the height of the season the truck receipts of peaches were 58 per cent of all peach receipts, apples 78 per cent, tomatoes 52 per cent, and peppers 16 per cent.

Some of the wholesale dealers do not like this tendency of change to motor-truck service because they can not hold the truck operator liable for injury to produce during the trip to market. Something may have to be done to provide insurance protection or direct liability on truck loads. Such protection in one way or another adds something to the costs of the trucking service. No other drawbacks from the point of view of the dealer are mentioned.

It is claimed that motor trucking to market helps the producers who are outside the old market-gardening

[1] Digested from article under the same title appearing in the Official Record, United States Department of Agriculture, August 1, 1928.

(Continued on page 128)

THE DESIGN OF PAVEMENT CONCRETE BY THE WATER-CEMENT RATIO METHOD

Reported by F. H. JACKSON, Senior Engineer of Tests, Division of Tests, United States Bureau of Public Roads

THE present method of specifying arbitrary proportions of cement, sand, and coarse aggregate for concrete, even though it may in most instances provide a satisfactory job from the standpoint of quality, is at best unsound from an economic point of view. This is true because in order to insure concrete of the designed strength under conditions which involve the possible use of a variety of materials it is necessary, when using fixed proportions, to adjust them on the basis of the most unfavorable combination possible under the specification. This is, of course, on the assumption that within the usual specification limits variations in such factors as character and grading of aggregates and quality of cement appreciably affect the quality of the concrete.

The investigation by the Bureau of Public Roads in cooperation with the New Jersey State Highway Commission,[1] as well as the tests now under way at Arlington, indicate that, in so far as character of aggregate is concerned, such variations may influence to a marked degree the transverse and tensile strength of the concrete, even though the crushing strength may be but slightly affected. Investigations conducted by the bureau in cooperation with the American Association of State Highway Officials indicate clearly that variations in the physical properties of Portland cements, all meeting the American Society for Testing Materials specifications, may quite appreciably affect the strength of the concrete.

From the standpoint of yield, also, it is well known that, under the present system of proportioning, variations in yield will occur, due to both type and gradation of aggregates. Such variations lead to fluctuations in the cement factor which are frequently the cause of misunderstandings and arguments between engineers and contractors. From many standpoints it seems desirable to so modify our procedure as to take advantage of such variations in aggregates and cement as normally occur in a given locality, as to produce concrete of the required strength at a minimum cost, and at the same time to provide such methods of handling and measuring the materials as will insure the production of fixed and uniform quantities of concrete. In this paper an attempt will be made to develop such a method by utilizing the well-established water-cement ratio law, and at the same time taking into consideration the various factors which render it impossible to make a general application of that law, as has been attempted by some authorities in the past.

The suggested method of design has already been tried with success on building construction, and no originality is claimed by the writer. This method will be developed, together with a discussion of changes in methods of securing bids on concrete-paving projects which, it is believed, will be necessary in order to make the suggested scheme of design effective.

WATER-CEMENT RATIO THEORY DISCUSSED

It is now almost universally recognized that there is a well-defined relation between the strength of concrete and the water-cement ratio for any given combination of materials. Many tests have also indicated that this relation when plotted takes a form which may be expressed by the general equation $S=\frac{A}{B^x}$. This is the general form of the well-known formula derived by Abrams.[2] Values for A and B depend upon the particular combination of materials used, as well as the character of the stress being investigated. S represents the strength of the concrete at 28 days and x an exponent, the water-cement ratio. To use the formula it is necessary to determine the constants for the particular materials being investigated, which, of course, must be done experimentally by testing a series of concrete specimens made with various water-cement ratios and plotting the strengths obtained against the corresponding ratios.

Before proceeding to a discussion of the principles governing the proposed method of designing concrete it may be well to state that, because of the experimental work involved, it will be necessary in the application of this method to have available a well-equipped laboratory with a qualified concrete testing engineer in charge. The designing of a concrete mixture to be used in a structure which is guaranteed to meet certain requirements as to strength, durability, etc., is just as much a technical operation requiring the services of a trained personnel as is the designing of the structure itself.

Furthermore, all attempts which have been made to design concrete through the application of certain formulas based only on considerations of grading of aggregates, such as fineness modulus, grading factor, surface area, etc., have failed, at least in so far as concrete for pavements is concerned, in one important respect—they do not take into account the character of the aggregates employed. By character is meant not only type—that is, crushed stone, gravel, etc.—but such factors as surface texture, angularity of fragments, etc. These factors affect the quality of the concrete in two ways—first, by influencing workability, which in turn controls the ratio of fine to coarse aggregate as well as the relative water content and, second, through the adhesion or bond which is produced between the cement and the aggregate surfaces.

The effects of such factors are particularly noticeable when the concrete is subjected to tensile and flexural stresses and are therefore of importance to the highway engineer. They apply alike to fine and coarse aggregates and explain why the experimental or trial method of design must be used. In other words, we have not yet reached the point where we can entirely discard actual tests of trial mixtures in favor of mathematical formulas in the design of concrete mixtures.

[1] Jackson, F. H., Comparative Tests of Crushed-Stone and Gravel Concrete in New Jersey, Public Roads, vol. 8, No. 12, February, 1928.

[2] Abrams, D. A., Design of Concrete Mixtures, Bull. 1, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

There are, however, certain fundamental principles underlying all methods of concrete design which must be thoroughly understood by everyone who intends to use the so-called trial method, and these will be discussed briefly before giving the various steps in the suggested method.

METHOD OF DETERMINING RATIO OF FINE TO COARSE AGGREGATE DESCRIBED

The first question to decide is the proper ratio in which to combine the various fine and coarse aggregates which are available for a given job, giving in each case due consideration to both workability and economy. There are four general rules which may be applied to this particular problem, as follows:

(1) The proportion of sand should be increased as the sand becomes coarser.

(2) The proportion of sand should be increased as the maximum size of the coarse aggregate becomes smaller.

(3) The proportion of sand should be increased as the percentage of fine material in the coarse aggregate becomes smaller.

(4) The proportion of sand should be increased as the percentage of angular fragments in the coarse aggregate becomes larger.

These principles are well known. The average specification for concrete, however, recognizes them only in a general way, usually by a clause giving the engineer the power to slightly change the proportion of fine to coarse aggregate to secure maximum density. It should be possible in designing the mix to fix this ratio much more accurately than is possible under the present arbitrary method. The most important point to remember is that a balance will have to be struck between a high sanded mix, which, although workable, is apt to be uneconomical, due to the fact that, for a constant water-cement ratio, more cement will be required for a given consistency, and a low sanded mix, which, although economical in so far as cement content is concerned, is apt to give trouble in placing.

In the writer's opinion the ideal combination is the one in which the voids in the coarse aggregate are maintained at a minimum, so as to permit the use of the smallest amount of mortar possible and still have a workable mix. In order to do this, the grading of the coarse aggregate must be controlled very carefully throughout the entire job, and this can best be done by handling and measuring it in separate sizes. This method serves also to eliminate segregation, and thus makes possible the use of a larger maximum size of coarse aggregate, which is economical from the standpoint of cement required. In the case of crushed stone the use of a larger size involves less crushing and is therefore more economical. Too little attention has been paid to such details in the past with the result that, although most of our concrete may be, and probably is, of satisfactory quality, it has not been designed so as to make the best use of a closely controlled coarse aggregate grading, which is the only way maximum workability can be attained with a minimum of cement. Just how far we can go in any particular case will depend, of course, upon the materials available, methods of finishing to be employed, etc. In general, there is no reason for using more mortar than is necessary to secure the proper finish. Under such a condition it will usually be found that there is enough mortar present to fill the voids in the coarse aggregate with a slight excess, and it is believed that, under our modern methods of finishing, this will prove sufficient in practically all cases to secure a dense, homogeneous concrete free from honeycomb.

For the usual run of materials for concrete roads the proper ratio of the volume of fine to course aggregate will range from a 30:70 ratio for a relatively fine sand combined with a closely graded easy-working coarse aggregate to a 40:60 ratio for a coarse sand combined with a high-void, harsh-working coarse aggregate. As previously stated, effort should be made when studying various possible combinations to keep as near the former ratio as possible, for the sake of economy, always remembering that the final value to use will depend entirely upon whether it is possible to secure a satisfactory finish and a concrete free from honeycomb with the placing and finishing equipment to be used on the job.

Using modern methods of handling it is believed that this condition can be attained frequently with a lower sand content than was possible with the old hand-finishing methods. As far as this factor is concerned, it is necessary to fall back upon judgment backed by actual observations on the job, rather than to rely entirely upon set formulas, helpful as they may be in giving preliminary indications.

As a guide, however, in making such a preliminary estimate of the proper ratio to use in any specific case, the values given in Table 1 may be taken. In general, these ratios are about the same as would be obtained by the use of the fineness modulus method suggested by the Portland Cement Association,[3] except that in no case is the percentage of fine aggregate less than 25 or more than 45 per cent of the sum of the volumes of the fine and coarse aggregate measured separately. It will be observed that these values illustrate the principles governing the proper ratio of fine to coarse aggregate

TABLE 1.—*Approximate ratios, by volume, of fine to coarse aggregate for paving concrete, machine finished*

Coarse aggregate size limits	Fine aggregate—size limits			
	0–No. 16	0–No. 8	0–No. 4	0–⅜-inch
No. 4 to ¾-inch	35:65	37:63	40:60	45:55
No. 4 to 1-inch	30:70	32:68	35:65	40:60
No. 4 to 2-inch	25:75	27:73	30:70	35:65

NOTES.—The above values are based on the use of the usual type of natural sand combined with a coarse aggregate consisting essentially of rounded fragments. With coarse aggregate consisting essentially of angular fragments it may be necessary to increase the percentage of sand slightly over the values above given.

It has been assumed that the concrete will be machine finished. For hand-finished work the percentage of sand may have to be increased somewhat.

For an aggregate to be given a certain maximum size, at least 15 per cent must be retained on the next smaller sieve shown in the table. For instance, a sand having 16 per cent retained on a No. 8 sieve is classed as a 0–No. 4 sand. For a sand to be classed as a 0–No. 16 sand, at least 15 per cent must be retained on a No. 30 sieve.

All coarse aggregates are assumed to be reasonably well graded from the maximum size to No. 4, with not more than 15 per cent passing the No. 4 sieve.

The use of a 0–No. 16 sand is not recommended, except under conditions where a coarser sand is not available, on account of the fact that concrete in which a fine sand is used is in general not quite so resistant to wear as when a coarse sand is used.

[3] Design and Control of Concrete Mixtures, published by the Portland Cement Association, Chicago, Ill.

given above. The values given in the table do not represent necessarily the final ratios to use. The best final values in any case can only be determined by trial, bearing in mind that the smallest amount of sand consistent with workability should ordinarily be used.

DETERMINATION OF PROPORTIONS BASED ON LABORATORY TESTS

Having tentatively fixed the proper ratio of fine to coarse aggregate for each available combination, the next step is to determine in each case how much cement and water to use to secure concrete meeting the requirements of the specification. For purposes of discussion it will be assumed that a minimum flexural strength (modulus of rupture) is specified. It is recognized that strength is not the only criterion upon which to judge the quality of concrete. Durability and resistance to wear are of great importance. However, it must be confessed that at present we are able to talk only in generalities with regard to durability. We have no definite requirements which may be set up in specifications. About all that we can do, assuming that the aggregates themselves possess the necessary properties as regards durability and resistance to wear, is to assume that concrete which will meet the strength requirements and which is sufficiently workable to be placed and finished in a satisfactory manner will be as permanent as it is possible to make it with our present knowledge of the art.

It will be assumed that the specifications call for a concrete which when tested under standard laboratory conditions will have a certain modulus of rupture, say 600 pounds per square inch at 28 days. The problem is to determine the most economical mixture which will give this strength and at the same time be sufficiently workable to place and finish properly on the job. Unfortunately at the present time it is impractical to attempt to control the quality of the cement to be used on the job further than to require that it pass the American Society for Testing Materials requirements. In trial determinations, therefore, it will be necessary to use a cement which corresponds to about the lowest-strength cement likely to be used on the job. The use in construction of any higher-strength cement than this simply serves to provide an additional factor of safety, in so far as the cement is concerned. Having selected the cement, it will now be necessary to fix experimentally the relation between the water-cement ratio and the flexural strength for this laboratory cement, using stock aggregates of known satisfactory quality. The determination of the strength developed at 28 days with water-cement ratios 0.6, 0.7, 0.8, 0.9, and 1.0 will usually give enough points from which to develop this relation. Such a relation for an assumed case is shown in Figure 1. It will be observed that a ratio of 0.7 gives a strength of approximately 600 pounds per square inch.

The next step is to make up concrete specimens with each of the aggregate combinations, using 0.7 water-cement ratio and the consistency which will be used on the job. It is important in this experiment to maintain the consistency as nearly constant as possible. With a constant water-cement ratio this will necessitate variable proportions, depending upon the type and gradation of the materials. The proper amount of cement to use in each case must be obtained by trial, adding small quantities of the aggregates in question to the cement paste until the proper consistency has been reached. The predetermined ratio between fine and coarse aggregate must be maintained throughout the operation. From the final quantities used the proportions either by weight or by volume may be readily calculated. The flow table[4] is recommended for determining relative consistency in the laboratory as being more positive than the slump test. It is, however, not a test for workability in the strict sense of the word, nothing having so far been developed to take the place of the eye in judging this important characteristic.

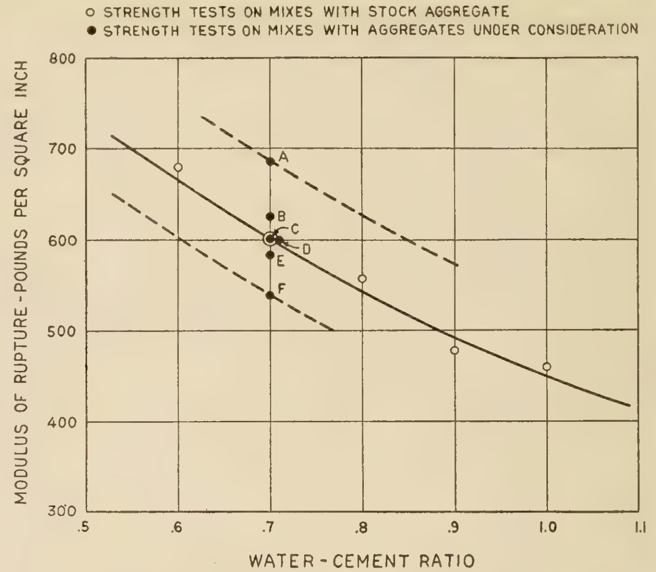


Fig. 1.—Typical Relation Between Water-Cement Ratio and Flexural Strength of Concrete at 28 Days

According to the original water-cement ratio theory, concrete specimens made with various aggregates as described above should all have substantially the same strength, because the water-cement ratios are the same and the mixtures are all workable. We know from experience, however, that the strengths will probably not be the same, due to the influence of the character of aggregate. Let us assume that the following strengths were actually obtained on six combinations of material:

Combination	Modulus of rupture in pounds per square inch	Combination	Modulus of rupture in pounds per square inch
A	670	D	600
B	625	E	580
C	600	F	535

These values are plotted in Figure 1. It is observed at once that four of the six combinations give strengths either identical or practically identical with the base or standard laboratory combination. There are, however, two outstanding exceptions, one much higher and one much lower. These two combinations, A and F, will be used as the basis for further discussion, since the same methods, somewhat simplified, can be applied to combinations B to E.

It is now assumed that had curves been developed for the relation between water-cement ratio and strength for these combinations, as was done for the base mix, the curves would be substantially parallel to the base

[4] A. S. T. M. Standards, vol. 2, 1927, p. 115.

curve over the comparatively narrow range in which we are interested. This may or may not be absolutely correct, but it is believed that for the range of mixtures covered by paving concrete it is substantially true. Granting this, we can omit the actual determination of this relationship for any of the combinations in which we are interested and simply draw through the value which we have plotted a line parallel to the basic curve. This has been done in Figure 1 for combinations A and F. To determine the water-cement ratio to use with either of these combinations to obtain a strength of 600 pounds, simply follow the curve representing the material either to the right or left, as the case may be, until it intersects the 600-pound line and use the corresponding water-cement ratio. Figure 1 shows this to be 0.85 for combination A and 0.60 for combination F. A choice between these combinations will depend entirely on which is the cheaper, all things considered, always assuming that the aggregates in both cases are structurally sound and have sufficient resistance to wear.

Before the cost can be determined it will be necessary to determine by trial method the proportions required in these two cases to give the consistency required at the water-cement ratios indicated—that is, 0.85 for combination A and 0.60 for combination F. Assume, for purposes of illustration, that the proportions for combination A with a 0.85 water-cement ratio reduce to 1:2:4 by volume and that the proportions for combination F with a 0.60 water-cement ratio reduce to 1:1½:3. Which of the two is the cheaper will, of course, depend almost entirely on the relative costs of the aggregates delivered on the job.

QUANTITIES OF MATERIAL DETERMINED BY SIMPLE CALCULATION

The next step is to work out for each case the theoretical cement factor as well as the quantities of aggregates required to produce a unit volume of concrete, knowing the specific gravities and weights per cubic foot of all of the materials. For this purpose a simple formula proposed by Stanton Walker[5] may be used. This formula gives the number of bags of cement required to produce 1 cubic yard of concrete, knowing the proportions as well as the weights per cubic foot and apparent specific gravities of the materials. It is based on the assumption that, for plastic mixes, the volume of concrete produced will be equal to the sum of the absolute volumes of cement and aggregates plus the volume of water, and may be expressed as follows:

$$C=\frac{27}{0.5+x+\frac{W_f}{62.4S_f}+\frac{W_g}{62.4S_g}}$$

where

C = number of bags of cement per cubic yard of concrete.
0.5 = approximate absolute volume of cement in one bag.
x = water-cement ratio = volume of water in a one-bag batch.
W_f = weight in pounds of fine aggregate used in a one-bag batch.
S_f = apparent specific gravity of fine aggregate.
W_g = weight in pounds of coarse aggregate used in a one-bag batch.
S_g = apparent specific gravity of coarse aggregate.
62.4 = weight per cubic foot of water.

This formula is sufficiently accurate for comparing concrete yields in the laboratory. It gives, in general, somewhat lower values for cement content than will be found by trial either in the laboratory or in the field. It will be necessary, before making final accurate estimates of quantities for field use, to make actual determinations of yield on the combination finally selected.

[5] Walker, Stanton, Estimating Quantities of Materials for Concrete, Bull. 1, National Sand and Gravel Association, Washington, D. C.

In the above formula it will be observed that the quantities

$$\frac{W_f}{62.4S_f} \text{ and } \frac{W_g}{62.4S_g}$$

represent, respectively, the absolute volumes of fine and coarse aggregate used with each bag of cement. The basis for the formula which is supported by laboratory determinations is that for given materials the absolute volumes of the aggregates used control the volume of concrete which will be obtained with a given amount of cement. This furnishes one of the principal arguments for measuring aggregates by weight instead of by volume, because as long as the specific gravity of the aggregate remains constant the weight of aggregate controls the yield irrespective of void content.

The application of the formula to the problem under discussion may now be made by continuing the illustration given above and assuming the following additional facts relative to the materials:

	Combination A	Combination F
Weight per cubic foot fine aggregate	90	80
Weight per cubic foot coarse aggregate	95	109
Apparent specific gravity of fine aggregate	2.65	2.65
Apparent specific gravity of coarse aggregate	2.55	2.70
Price per ton delivered, fine aggregate	$1.75	$1.00
Price per ton delivered, coarse aggregate	$2.00	$1.00
Price per bag, cement	$0.60	$0.60

These values, though assumed, might readily be encountered in actual practice, the idea in this case being that the aggregates represented by combination A are from sources some distance from the work, so that the cost of transportation must be considered, whereas aggregates F are locally available. Any other factor which might cause differences in price, such as cost of production, might, of course, just as well have been assumed.

Applying the data given in the table to the formula and remembering that a water-cement ratio of 0.85 and proportions of 1:2:4 have been selected for combination A and corresponding values of 0.60 and 1:1½:3 for combination B, the cost of materials for each combination is obtained as follows:

ESTIMATE FOR COMBINATION A

$$C=\frac{27}{0.5+0.85+\frac{180}{62.4\times2.65}+\frac{380}{62.4\times2.65}}=5.6 \text{ bags of cement}$$

Weight of fine aggregate per cubic yard of concrete $=\frac{5.6\times180}{2{,}000}=0.505$ tons.

Weight of coarse aggregate per cubic yard of concrete $=\frac{5.6\times380}{2{,}000}=1.065$ tons.

COSTS

Cement, 5.6 bags, at $0.60	$3.36
Fine aggregate, 0.505 ton, at $1.75	.88
Coarse aggregate, 1.065 tons, at $2	2.13
Cost of materials per cubic yard of concrete	6.37

ESTIMATE FOR COMBINATION F

$$C=\frac{27}{0.5+0.60+\frac{120}{62.4\times 2.65}+\frac{327}{62.4\times 2.70}}=7.16 \text{ bags of cement}$$

Weight of fine aggregate per cubic yard of concrete $=\frac{7.16\times 120}{2{,}000}=0.429$ ton.

Weight of coarse aggregate per cubic yard of concrete $=\frac{7.16\times 327}{2{,}000}=1.18$ tons.

COSTS

Cement, 7.16 bags, at $0.60	$4.30
Fine aggregate, 0.429 ton, at $1	.43
Coarse aggregate, 1.17 tons, at $1	1.18
Cost of materials per cubic yard of concrete	5.91

These values are not given as typical of the relative cost of concrete using imported or local aggregates, but only to illustrate a method whereby reliable information as to comparative costs may be obtained, as well as to show that the most expensive concrete is not necessarily the one containing the most cement.

There are other factors in addition to actual cost which must be considered when comparing sources of aggregate supply. For instance, the selection of materials represented by combination F in the above illustration would only be justified on the basis of an adequate supply of material equal in quality and of the same grading as the sample upon which the design is based. This, in turn, involves not only a thorough inspection of the source as to the extent and uniformity of the deposit but also presupposes adequate plant equipment for producing the aggregates. It is wasted effort to go to the trouble of designing a mix based on an examination of samples submitted for test and then find that it is either impossible or impractical to produce materials equal to the samples for the actual job. In the past most of the attempts to design concrete mixtures for paving work using local materials have failed because proper emphasis was not placed on the importance of uniformity of the material supply. Uniform concrete may be obtained in no other way.

EFFECT ON SPECIFICATIONS DISCUSSED

In using the proposed method of designing concrete mixtures in actual construction, it will be necessary to change the present method of specifying arbitrary proportions to a specification based on a certain required minimum strength. Such a method of specifying has recently been suggested by J. T. Voshell, district engineer, Bureau of Public Roads, which would also involve a change in the method of bidding. Application of the proposed method of design can be worked out as follows:

Each bidder, instead of specifying a price per square yard for concrete in place, would be required to submit separate bids for all materials which he is prepared to furnish, together with a separate bid price per square yard for mixing, placing, finishing, and curing the concrete in accordance with the requirements of the specification. After receipt of proposals the engineer will examine all of the sources proposed, first, with the view to eliminating any which do not comply with the basic requirements of the specifications, and, second, in order to determine which of the materials proposed will produce concrete of the required quality at the lowest cost, using a procedure similar to that outlined above. The award should be made to the contractor who can supply the materials and mix and place the concrete at the lowest total price per square yard.

With this method of procedure the responsibility for selecting the materials and adjusting the mix to secure concrete of the desired quality, as well as the responsibility of seeing that the production of the concrete is carried out in accordance with the specifications, rests solely with the engineer. This is where it belongs, unless we are prepared to go to the other extreme and specify the quality of the finished concrete and allow the contractor to use any materials and methods of production he desires so long as he fulfills this requirement. The writer believes that we should adhere strictly to one course or the other and not attempt to control every step in the process of construction and still hold the contractor responsible for the result.

In applying the method of design described in this paper the selection of aggregates and proportions to be employed should be based on laboratory tests under controlled conditions so as to insure that when the pavement is constructed using the same aggregates and proportions and in accordance with the detailed specifications governing mixing, placing, and curing a structure of satisfactory quality will result. It is believed that under such circumstances specifications should not contain provisions as to the strength of the finished concrete, and strength tests, if made, should be for the guidance of the engineer only.

If, on the other hand, it is desired to specify the quality of finished product only, the same technical procedure for designing mixtures may still be employed, only in this case it becomes a method to be applied by the contractor instead of the engineer, because under such a specification the contractor must determine for himself the materials to be used and the proportion in which to combine them in order to make an intelligent bid.

TRUCK IS A BIG FACTOR IN FRUIT TRANSPORT

(Continued from page 123)

region, giving them the advantages of prompt delivery wherever the load is wanted and without delay, rehandling, or extra charges.

Many of the dealers have their own trucks which they take to the producing section and do their buying direct. This plan suits the grower rather well because he sells for cash and need have no more trouble about the matter.

As for the condition of the trucked produce, there is some discussion about the effect on soft fruits, yet it is claimed that strawberries trucked to market hold their condition better and longer than those shipped in iced cars, although the iced berries will look better for the first four or five hours.

O

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.

DEPARTMENT BULLETINS

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257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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387D. Public Road Mileage and Revenues in the Southern States, 1914.
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DEPARTMENT BULLETINS—Continued

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DEPARTMENT CIRCULARS

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331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

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93M. Direct Production Costs of Broken Stone.
*105M. Federal Legislation Providing for Federal Aid in Highway Construction and the Construction of National Forest Roads and Trails. 5c.

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*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.
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TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION AS OF JULY 31, 1928

State	Completed Mileage	Under Construction: Estimated Total Cost	Under Construction: Federal Aid Allotted	Under Construction: Mileage: Initial	Under Construction: Mileage: Stage	Under Construction: Mileage: Total	Approved for Construction: Estimated Total Cost	Approved for Construction: Federal Aid Allotted	Approved for Construction: Mileage: Initial	Approved for Construction: Mileage: Stage	Approved for Construction: Mileage: Total	Balance of Federal Aid Funds Available for New Projects	State
Alabama	1,754.8	$ 5,469,983.89	$ 2,719,928.34	299.4	55.9	355.3	$ 423,504.73	$ 211,439.32	27.4	12.4	39.8	$ 1,605,326.53	Alabama
Arizona	870.5	1,465,014.16	1,247,575.75	66.8	0.6	67.4	122,094.33	42,024.80		4.2	4.2	2,896,024.65	Arizona
Arkansas	1,678.2	4,888,177.23	2,167,222.99	180.7		180.7	246,306.85	116,258.35	9.9	6.6	16.5	1,769,630.36	Arkansas
California	1,455.9	7,027,478.29	3,224,918.38	163.4	8.3	171.7	1,043,058.58	439,979.93	16.8		16.8	3,268,277.30	California
Colorado	973.4	5,005,733.20	2,595,332.93	189.0	23.6	212.6	668,950.84	343,365.28	26.4		26.4	2,277,348.10	Colorado
Connecticut	206.5	3,269,777.06	831,098.01	34.4		34.4	285,289.67	66,951.17	3.6		3.6	566,752.61	Connecticut
Delaware	195.7	470,022.22	95,739.75	5.7	3.9	9.6	421,823.60	196,095.80	15.6		15.6	149,880.44	Delaware
Florida	385.7	4,185,912.27	1,773,093.63	99.7	5.5	105.2	905,396.31	333,505.44	30.6		30.6	1,211,619.16	Florida
Georgia	2,446.6	4,034,460.07	1,941,333.15	173.0	41.6	214.6	2,574,450.59	1,035,137.76	107.9	29.2	137.1	18,200.47	Georgia
Idaho	953.5	1,907,130.88	1,134,135.23	103.3	53.4	156.7	1,252,796.89	740,083.22	112.3	13.0	125.3	94,935.97	Idaho
Illinois	1,692.6	20,188,584.26	9,280,164.08	622.9		622.9	3,665,454.99	1,826,435.15	138.1		138.1	67,033.69	Illinois
Indiana	1,060.1	10,811,333.26	5,156,644.86	330.2	3.5	333.7	1,195,527.30	572,109.27	44.4		44.4	260,765.41	Indiana
Iowa	2,865.2	5,945,294.22	2,509,472.52	121.0	127.7	248.7	1,827,757.31	778,011.52	10.3	71.4	81.7	171,307.77	Iowa
Kansas	2,212.9	5,109,109.56	2,030,143.66	255.2		255.2	2,748,885.59	1,083,858.77	280.4	6.5	286.9	658,462.77	Kansas
Kentucky	1,156.8	4,966,935.31	2,459,328.98	225.4		225.4	1,590,138.84	793,234.17	86.6		86.6	279,233.69	Kentucky
Louisiana	1,276.0	4,149,931.86	2,067,207.29	193.0		193.0	724,325.88	239,803.83	8.8		8.8	337,598.06	Louisiana
Maine	428.7	1,294,982.17	528,952.60	39.7		39.7	1,444,616.78	432,361.22	31.6		31.6	1,149,317.64	Maine
Maryland	564.8	557,576.12	269,630.00	22.6		22.6	1,221,024.60	521,645.00	47.7	7.2	54.9	39,071.23	Maryland
Massachusetts	505.6	3,811,948.21	1,138,407.82	71.7		71.7	813,921.57	223,490.72	15.0	6.5	21.5	1,944,710.30	Massachusetts
Michigan	1,343.3	3,380,003.46	5,673,143.08	338.6		338.6	1,049,701.20	478,530.00	29.5		29.5	529,249.95	Michigan
Minnesota	3,823.8	6,757,547.04	2,250,100.00	343.2	54.7	397.9	730,758.07	129,000.00	12.5	20.6	33.1	398,471.43	Minnesota
Mississippi	1,533.6	4,398,804.92	2,168,480.79	228.1	30.9	259.0	1,299,666.39	414,544.23	30.4	0.6	31.0	559,311.54	Mississippi
Missouri	2,210.0	4,959,964.86	2,027,798.28	145.3	39.0	184.3	1,475,482.98	625,767.40	39.3	18.7	58.0	1,419,542.68	Missouri
Montana	1,303.1	4,295,996.74	2,808,194.67	356.0	7.4	363.4	1,492,961.27	821,945.23	147.6	7.4	155.0	4,354,131.45	Montana
Nebraska	3,103.7	6,021,041.68	2,999,276.83	608.3	161.5	769.8	176,729.15	88,364.57	0.1	23.1	23.2	1,978,356.91	Nebraska
Nevada	1,010.1	1,188,333.63	1,040,950.45	137.3	36.9	174.2	125,607.95	109,902.20		41.1	41.1	514,111.19	Nevada
New Hampshire	305.7	715,499.39	294,415.16	20.4		20.4	436,850.71	168,275.78	7.8		7.8	65,727.25	New Hampshire
New Jersey	426.6	5,215,438.03	915,402.35	63.0		63.0						253,177.00	New Jersey
New Mexico	1,752.3	2,420,333.20	1,607,357.28	143.2	0.5	143.7	912,501.97	578,124.43	70.9		70.9	680,228.45	New Mexico
New York	1,868.3	41,915,000.00	7,489,393.95	483.6		483.6	7,894,072.00	1,645,597.50	106.1	8.6	114.7	3,677,302.65	New York
North Carolina	1,599.2	1,671,666.10	798,437.88	72.1	13.0	85.1	664,295.97	324,400.00	11.2	19.6	30.8	1,084,145.16	North Carolina
North Dakota	3,197.1	3,504,500.48	1,644,804.15	623.3	167.4	790.7	1,059,931.41	425,431.28	164.5	75.8	240.3	587,548.37	North Dakota
Ohio	1,824.3	11,960,386.74	4,217,843.32	250.1	6.0	256.1	4,725,672.10	1,527,156.04	102.7	12.7	115.4	2,106,977.07	Ohio
Oklahoma	1,591.1	3,201,431.64	1,532,062.76	169.5	6.4	175.9	1,805,584.80	806,386.50	107.2	15.5	122.7	409,181.44	Oklahoma
Oregon	1,104.9	1,517,571.16	847,554.19	40.7		40.7	342,895.30	186,863.05	18.8		18.8	1,226,909.14	Oregon
Pennsylvania	1,846.7	13,493,570.61	3,826,489.58	236.4		236.4	5,314,598.47	1,658,275.74	103.8		103.8	1,421,539.75	Pennsylvania
Rhode Island	137.8	1,595,034.57	406,914.92	25.2		25.2	311,081.95	80,919.55	4.0		4.0	576,046.16	Rhode Island
South Carolina	1,639.7	8,259,127.48	1,827,138.22	197.4	120.7	318.1	394,006.44	69,700.00	10.4	8.2	18.6	64,396.43	South Carolina
South Dakota	2,987.8	2,997,515.89	1,604,699.81	483.1	55.0	538.1	379,661.52	208,813.74	72.8	42.1	114.9	443,253.50	South Dakota
Tennessee	1,075.6	4,524,754.78	1,939,405.06	139.0		139.0	4,267,517.00	1,368,324.58	25.6	94.3	119.9	254,777.15	Tennessee
Texas	5,997.1	8,799,695.82	3,507,392.65	206.0	125.1	331.1	6,730,764.34	2,842,089.38	222.2	165.7	387.9	3,563,138.66	Texas
Utah	833.0	1,803,880.93	1,221,511.17	92.7	5.3	98.0	386,634.05	277,669.18	22.9	1.5	24.4	184,679.58	Utah
Vermont	201.3	2,213,149.03	571,487.97	48.9		48.9	584,323.08	147,454.36	11.9		11.9	38,964.23	Vermont
Virginia	1,266.9	4,345,329.44	1,360,972.12	99.3	21.6	120.9	1,372,089.58	313,899.48	55.8		55.8	57,540.67	Virginia
Washington	777.7	4,105,232.53	1,403,000.00	105.4	18.1	123.5	1,142,840.68	440,236.89	21.2	5.0	26.2	515,843.46	Washington
West Virginia	606.9	2,781,092.00	1,240,175.25	105.7		105.7	1,239,058.83	539,355.57	39.6	12.4	52.0	237,849.40	West Virginia
Wisconsin	2,044.1	8,683,308.50	3,456,798.23	297.7	31.0	328.7	649,126.58	166,625.24	14.5	10.0	24.5	1,450,278.65	Wisconsin
Wyoming	1,453.5	2,348,700.43	1,474,109.77	237.5	32.1	269.6	377,302.98	240,855.91	47.9		47.9	32,239.73	Wyoming
Hawaii	36.2	301,973.75	60,383.43	3.2		3.2	175,931.99	57,501.20	1.8		1.8	1,064,241.58	Hawaii
TOTALS	71,584.9	263,935,269.07	105,386,023.29	9,497.3	1,256.6	10,753.9	70,692,974.01	26,737,799.75	2,516.4	739.9	3,256.3	48,514,656.78	TOTALS

The term stage construction refers to additional work done on projects previously improved with Federal-aid. In general, such additional work consists of the construction of surface of higher type than was provided in the original improvement.